

**IMPACT OF AASHTO LRFD BRIDGE DESIGN  
SPECIFICATIONS ON THE DESIGN OF TYPE C AND AASHTO  
TYPE IV GIRDER BRIDGES**

A Thesis

by

**SAFIUDDIN ADIL MOHAMMED**

Submitted to the Office of Graduate Studies of  
Texas A&M University  
in partial fulfillment of the requirements for the degree of

**MASTER OF SCIENCE**

December 2005

Major Subject: Civil Engineering

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Approved by:

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Major Subject: Civil Engineering

## ABSTRACT

Impact of AASHTO LRFD Bridge Design Specifications on the Design of Type C and  
AASHTO Type IV Girder Bridges. (December 2005)

Safiuddin Adil Mohammed, B.E., Osmania University

Chair of Advisory Committee: Dr. Mary Beth D. Hueste

This research study is aimed at assisting the Texas Department of Transportation (TxDOT) in making a transition from the use of the *AASHTO Standard Specifications for Highway Bridges* to the *AASHTO LRFD Bridge Design Specifications* for the design of prestressed concrete bridges. It was identified that Type C and AASHTO Type IV are among the most common girder types used by TxDOT for prestressed concrete bridges. This study is specific to these two types of bridges. Guidelines are provided to tailor TxDOT's design practices to meet the requirements of the LRFD Specifications.

Detailed design examples for an AASHTO Type IV girder using both the AASHTO Standard Specifications and AASHTO LRFD Specifications are developed and compared. These examples will serve as a reference for TxDOT bridge design engineers. A parametric study for AASHTO Type IV and Type C girders is conducted using span length, girder spacing, and strand diameter as the major parameters that are varied. Based on the results obtained from the parametric study, two critical areas are identified where significant changes in design results are observed when comparing Standard and LRFD designs. The critical areas are the transverse shear requirements and interface shear requirements, and these are further investigated.

The interface shear reinforcement requirements are observed to increase significantly when the LRFD Specifications are used for design. New provisions for interface shear design that have been proposed to be included in the LRFD Specifications in 2007 were evaluated. It was observed that the proposed interface shear provisions will significantly reduce the difference between the interface shear reinforcement requirements for corresponding Standard and LRFD designs.

The transverse shear reinforcement requirements are found to be varying marginally in some cases and significantly in most of the cases when comparing LRFD designs to Standard designs. The variation in the transverse shear reinforcement requirement is attributed to differences in the shear models used in the two specifications. The LRFD Specifications use a variable truss analogy based on the Modified Compression Field Theory (MCFT). The Standard Specifications use a constant 45-degree truss analogy method for its shear design provisions. The two methodologies are compared and major differences are noted.

## **DEDICATION**

To all the Civil Engineers who are striving to make this world a better place to live.

## **ACKNOWLEDGMENTS**

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Last but not the least, thanks are due to my family, especially my father, Dr. Shaik Chand, and mother, Naseem Sultana, for their love, encouragement, and support.

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# 1. INTRODUCTION

## 1.1 BACKGROUND AND PROBLEM STATEMENT

Bridge structures constructed across the nation not only require the desired safety reserve, but also consistency and uniformity in the level of safety. This uniformity is made possible using improved design techniques based on probabilistic theories. One of such techniques is reliability based design, which accounts for the inherent variability of the loads and resistances to provide uniform safety of the structure. The level of safety is measured in terms of a reliability index.

The American Association of State Highway and Transportation Officials (AASHTO) first introduced the *Standard Specifications for Highway Bridges* in 1931 and since then these specifications have been updated through 17 editions, with the latest edition being published in 2002 (AASHTO 2002). The *AASHTO Standard Specifications for Highway Bridges* were based on the Allowable Stress Design (ASD) philosophy until 1970, after which the Load Factor Design (LFD) philosophy was incorporated in the specifications. These methodologies provide the desirable level of safety for bridge designs, but do not ensure uniformity in the level of safety for various bridge types and configurations.

To bring consistency in the safety levels of bridges, AASHTO introduced the *AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications* in 1994 (AASHTO 1994). These specifications were calibrated using structural reliability techniques that employ probability theory.



The *AASHTO LRFD Bridge Design Specifications* (AASHTO 2004) are intended to replace the latest edition of the *AASHTO Standard Specifications for Highway Bridges* (AASHTO 2002), which will not continue to be updated except for corrections. The Federal Highway Association (FHWA) has mandated that this transition be completed by State Departments of Transportation (DOTs) by 2007. The design philosophy adopted in the *AASHTO LRFD Bridge Design Specifications* provides a common framework for the design of structures made of steel, concrete and other materials.

Many state DOTs within the US have already implemented the AASHTO LRFD Specifications for their bridge designs and the remaining states are transitioning from the Standard Specifications to the LRFD Specifications. The fact that many bridge engineers are not very familiar with reliability based design and new design methodologies adopted in the LRFD Specifications can potentially slow down the process of transition to LRFD based design. This study is aimed towards helping bridge engineers understand and implement AASHTO LRFD bridge design for prestressed concrete bridges, specifically Type C and AASHTO Type IV girder bridges.

The Texas DOT (TxDOT) is currently using the *AASHTO Standard Specifications for Highway Bridges* with slight modifications for designing prestressed concrete bridges. TxDOT is planning to replace the AASHTO Standard Specifications with the AASHTO LRFD Specifications for their bridge design. This study will provide useful information to aid in this transition, including guidelines and detailed design examples. The impact of using the LRFD Specifications on the design of prestressed concrete bridge girders for various limit states is evaluated using a detailed parametric study. Issues pertaining to the design and the areas where major differences occur are identified and guidelines addressing these issues are suggested for adoption and implementation by TxDOT.

## 1.2 OBJECTIVES AND SCOPE

The main purpose of this research study is to develop guidelines to help TxDOT adopt and implement the *AASHTO LRFD Bridge Design Specifications*. The impact of the AASHTO LRFD Specifications on different design limit states is quantified. The objectives of this study are as follows

1. Identify major differences between the AASHTO Standard and LRFD Specifications.
2. Generate detailed design examples based on the AASHTO Standard and LRFD Specifications as a reference for bridge engineers to follow for step by step design and highlight major differences in the designs.
3. Evaluate the simplifying assumptions made by TxDOT for bridge design for their applicability when using the AASHTO LRFD Specifications.
4. Conduct a parametric study based on parameters representative of Texas bridges to investigate the impact of the AASHTO LRFD Specifications on the design as compared to the AASHTO Standard Specifications.
5. Identify the areas where major differences occur in the design and develop guidelines on these critical design issues to help in implementation of the LRFD Specifications by bridge engineers.

This study focuses on Type C and AASHTO Type IV prestressed concrete bridge girders, which are widely used in the state of Texas and other states.

## 1.3 RESEARCH PLAN

The following seven major tasks were performed to accomplish the objectives of this research study

### *Task1: Literature Review*

The previous studies related to the development and implementation of the AASHTO LRFD Bridge Design Specifications have been reviewed in detail. The main focus is on reliability theory and the difference between reliability based designs and the

designs based on other methodologies employed in the Standard Specifications. The literature review discusses the studies related to the development of dead load, live load, dynamic load models and distribution factors. The studies that form the basis of new methodologies employed in the LRFD Specifications for transverse and interface shear designs are also reviewed. The past research evaluating the impact of the AASHTO LRFD Bridge Design Specifications on bridge design as compared to the AASHTO Standard Specifications is also included in the literature review. The observations made from the review of the relevant literature are documented in a concise manner.

### ***Task 2: Development of Detailed Design Examples***

Detailed design examples for an AASHTO Type IV girder bridge were developed using the *AASHTO Standard Specifications for Highway Bridges, 17<sup>th</sup> edition* (2002) and the *AASHTO LRFD Bridge Design Specifications, 3<sup>rd</sup> edition* (2004). An AASHTO Type IV girder bridge was selected for detailed design comparison as this is widely used by TxDOT. Type C girder bridges are also used in many cases, but the design process does not differ significantly from that of AASHTO Type IV girder bridges.

The following parameters, based on TxDOT's input, were considered for this detailed design example: span length = 110 ft., girder spacing = 8 ft., strand diameter = 0.5 in., deck thickness = 8 in., wearing surface thickness = 1.5 in., skew = 0° and relative humidity = 60%. The live load for the Standard design is taken as HS20-44, whereas for LRFD design it is HL-93. T501 type railings were used for the design. The limit states considered for the design are service bending stress, flexural strength, fatigue, shear and deflection. All the applicable load combinations except that of extreme events were taken into account. Concrete strengths at release and at service were optimized consistent with TxDOT's methodology. Camber was calculated using the Hyperbolic Functions method used by TxDOT.

The detailed design examples highlight major differences in the AASHTO Standard and LRFD design methodologies. A table illustrating the percent difference in each design parameter is presented at the end of the detailed design examples. These examples are aimed to be comprehensive and easy to follow in order to provide a good reference for bridge engineers.

### ***Task 3: Review of TxDOT Design Criteria for Bridge Design***

Simplifying assumptions made by TxDOT in bridge design were evaluated for their applicability when using the AASHTO LRFD Specifications. The simplifications considered for evaluation include the assumption of the modular ratio between slab and beam concrete to be unity throughout the design. In addition the practice of not updating the modular ratio for calculating actual prestress losses, flexural strength limit state checks and deflection calculations was assessed. The impact of these simplifications in LRFD design were conveyed to TxDOT during this project and, based on their input, design procedures were finalized. The modifications in the designs or deviations from the LRFD Specifications to simplify the design are clearly stated and their limitations are illustrated.

### ***Task 4: Identification of Critical Parameters***

A parametric study was conducted to assess the impact of utilizing the current LRFD Specifications (AASHTO 2004) on the design of Type C and AASHTO Type IV girder bridges as compared to the current Standard Specifications (AASHTO 2002). The main parameters for this study were girder spacing, span length, concrete strengths at release and at service, skew angle, and strand diameter. The values for these parameters were chosen based on TxDOT's input such that they are representative of the typical bridges in Texas. The concrete strengths at service and at release were limited to values commonly available from Texas precasters. The spans and girder spacing are dictated by TxDOT practice. Typically in TxDOT designs, all the girders in the bridge are designed

as interior girders. Following this practice, only interior girders were considered for this parametric study.

#### ***Task 5: Parametric Study***

A design program was developed to facilitate the design of Type C and AASHTO Type IV girder bridges for different parameters using the AASHTO Standard and LRFD Specifications. The program handles all the limit states considered for the detailed design examples. Prestress losses were calculated using TxDOT's methodology for Standard design and using the AASHTO LRFD Specifications for LRFD design. Concrete strengths at service and at release were optimized following an iteration process used by TxDOT. The flexural strength was evaluated based on the actual concrete strength when determining the transformed effective slab width. The transverse reinforcement is based on the demand of both transverse and interface shears. The results of the parametric study were verified using TxDOT's bridge design software PSTRS14 (TxDOT 2004) results. The results are presented in tabular and graphical formats to highlight the major differences in the designs using the Standard and LRFD Specifications.

#### ***Task 6: Identification of Critical Design Issues***

Two major areas requiring further study were identified based on the detailed design examples and the results of the parametric study. Transverse shear design was identified because considerable changes took place when the AASHTO LRFD Specifications adopted a significantly different methodology for shear design. The shear design in the Standard Specifications is based on a constant 45-degree truss analogy for shear, whereas the LRFD Specifications uses a variable truss analogy based on Modified Compression Field Theory (MCFT) for its shear provisions. A second area identified for further study is the interface shear design for which the LRFD Specifications gives new formulas based on recent results from studies in this area.

***Task 7: Guidelines for Critical Design Issues***

Detailed study on the background of interface and transverse shear was conducted. Additional guidelines for these design issues are provided so that smooth transitioning to the AASHTO LRFD Specifications is made possible. The recent studies in the respective areas were reviewed and the findings are noted. For example it is anticipated that new provisions for interface shear design will be presented in 2005 by a committee for approval and inclusion in the LRFD Specifications. The impact of the new provisions on the interface shear design was studied and recommendations are provided. A design example for the recommended revised design criteria is provided.

**1.4 OUTLINE**

Section 1 provides an introduction to this research study. Section 2 includes the documentation of the literature review. Section 3 highlights the TxDOT practices and the simplifications in the design made by TxDOT. The impact of these simplifications on the critical design parameters is presented. Section 4 provides the outline and the methodology of the parametric study. Section 5 and 6 presents the results of the parametric study conducted for AASHTO Type IV and Type C girders respectively. Section 7 presents the background on shear design issues and recommendations. Section 8 outlines the summary of the study, conclusions and recommendations for future research. Detailed design examples are included as Appendix A.

## **2. LITERATURE SURVEY**

### **2.1 GENERAL**

The topics that formed the basis for the development of the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2004) are reviewed in this section. This includes studies related to development of live load, dead load, and dynamic load models for bridge design; formulation of load distribution factors for prestressed concrete girder bridge design, development of resistance models for prestressed concrete girder bridge design; and reliability theory. A review of the comparison of the LRFD and Standard Specifications carried out by Hueste et. al. (2003) and Richard et. al. (2002) has been included in this section. The effect of the LRFD Specifications on the design of bridges found by previous studies is also reviewed briefly.

### **2.2 COMPARISON OF AASHTO STANDARD AND LRFD SPECIFICATIONS**

#### **2.2.1 General**

The load and resistance factors found in the LRFD Specifications are based on the theory of reliability and the specifications are modified to overcome the shortcomings observed in the Standard Specifications. Live load models based on the model truck traffic on bridges with various span lengths were developed to properly calibrate the LRFD Specifications, which resulted in many changes in the load combinations for various limit states. The Standard Specifications used the HS-20 load model, which did not prove adequate to model actual traffic loading. A new live load model, HL-93, with refined distribution factors is proposed in the LRFD Specifications. The service load design method, although easy to understand and apply, does assign load factors for different types of loads. Instead the effect of loads are restricted to a fraction of the yield stress, modulus of rupture, buckling, or crushing load, which causes non-uniformity in the safety. The Load Factor Design (LFD) takes into account the effect of factored loads but does not account for uniform safety. The Load and Resistance Factor

Design (LRFD) method accounts for the inherent variability of loads and resistances using reliability theory. Loads and resistances are treated as random variables and the calibration process aims at minimizing the area of overlap where the load is greater than resistance. The safety of a structure is measured in terms of a reliability index and is compared to the selected target reliability index (Mertz et al. 1996).

The AASHTO Standard Specifications are based on the Allowable Stress Design (ASD) and LFD philosophies, whereas the LRFD Specifications are based on probability-based limit state philosophy. Hueste et. al. (2003) presented a detailed comparison between the LRFD and Standard Specifications. Richard et. al. (2002) also compared the two specifications using detailed examples. Some of the significant differences between the two specifications are listed below.

### **2.2.2 Significant Changes**

The Standard Specifications express the impact factor as a fraction of live load and a function of span length as  $I = 50/(L+125)$ , where  $I$  is the impact factor and  $L$  is the length of the span in feet. Therefore for a span of 100 ft. the value of  $I$  is 0.22. The LRFD Specifications give a constant value of impact factor depending on the components and limit state under consideration. For instance, the impact factor for girder design for limit states other than the fatigue and fracture limit states comes out to be 0.33 (33% increase in the truck load only).

The LRFD Specifications allow the use of refined analysis for the determination of live load distribution factors (DFs) whereas the Standard Specifications gives simple expressions for the live load distribution to exterior and interior girders. For common bridge types, the LRFD Specifications includes an approximate method, based on parametric analyses of selected bridge geometries. This method can be used only if the bridge geometry falls within the limits of the parametric analysis for which the DF equations are based. The LRFD Specifications specify reduction factors for application to live load moment and shear to account for the skew of the bridge. The skew factor for moment decreases the moment distribution factor for interior and exterior girders for



certain angles. The skew factor for shear increases the shear distribution factor for the interior and exterior girders at the obtuse corners of the skewed bridge. The overhang distance is limited as per the Articles 4.6.2.2.1 and 4.6.2.2.2 of the LRFD Specifications.

The LRFD Specifications provide three different options for the estimation of time dependent prestress losses. The options are lump sum estimates, refined estimates and exact estimates using time-step method. Expressions are provided for the lump-sum estimate of the time dependent prestress losses for different type of bridges. The lump sum time dependent losses are based on the compressive strength of concrete and the partial prestressing ratio. The Standard Specifications provides the option of lump-sum method and refined method for the estimation of time-dependent losses. The lump sum estimates are given as specific values for two different values of concrete strength at service.

### **2.2.3 Limit States for Prestressed Concrete Bridge Girders**

The check for compressive stress in the prestressed concrete girder using Service I limit state with a live load factor of 1.0, tensile stress check using Strength III limit state with a live load factor of 0.8 is specified in LRFD Specifications. The Standard Specifications specifies the Group I loading for service limit state with a load factor of 1.0.

The calibration of the LRFD Specifications was focused on the ultimate limit states but is not readily applicable to other design considerations traditionally evaluated using service loads, such as stress limits, deflections, and fatigue. This difference accounts for the establishment of the Service III limit state for prestressed concrete structures, which evaluates the tensile stress in the structure, with the objective of crack control in prestressed concrete members. The load and resistance factors for limit states other than the strength limit states were selected to provide designs that are consistent with the Standard Specifications. A larger number of limit states must be accounted for in design using the LRFD Specifications, and the extreme load cases such as collision forces has to be included if their occurrence is possible in the design life of the bridge.

### 2.3 RELIABILITY OF PRESTRESSED CONCRETE BRIDGE GIRDERS

The calibration of the LRFD Specifications was based on reliability theory of analysis. This method of analysis, which has evolved over the years, employs the probability of failure as the design criteria. The load and resistance factors are chosen such that the safety of a structure against the loads is at a prescribed level, called the target safety level. The LRFD Specifications describe the target safety levels for different structures based on various criteria. Nowak et. al. (1996) discuss the selection of a optimum target safety level for bridges for various limit states. The optimum safety level depends on several factors such as consequences of failure and cost of safety. Increasing the safety of any structure is desirable, but this increases the cost of construction and requires that the safety of a structure be restricted to a certain level in order to render a safe and economic design. Figure 2.1 shows the relationship between the cost of failure and the reliability index  $\beta$ . The increase in the reliability index,  $\beta_T$  reduces the cost of failure ( $C_F$ ) and the probability of failure ( $P_F$ ), and increases the cost of investment ( $C_I$ ). The total cost ( $C_T$ ) is the sum of cost of failure and the cost of investment.

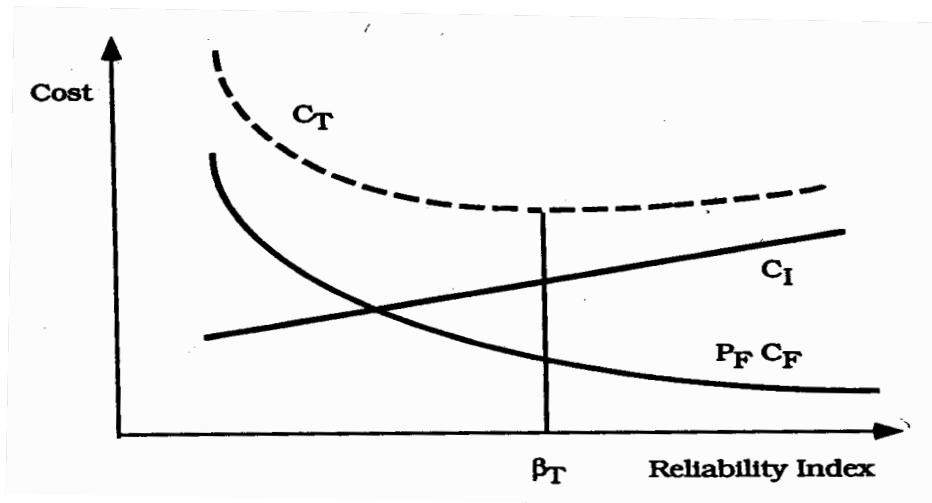


Figure 2.1. Cost vs. Reliability Index and Optimum Safety Level (Nowak et al. 1996)

The load and resistance parameters can be treated as random variables due the randomness in the frequency of occurrence, magnitude of live loads, material properties, dimensions, and geometries. The parameters available from the statistical models for load and resistance for highway bridges proposed by Nowak (1993a, 1995), Tabsh and Nowak (1992), Nowak and Hong (1992) and Hwang and Nowak (1992) were used for reliability analysis. Of the various limit states associated with structural failure, the ultimate limit states are related to loss of load carrying capacity, such as flexural strength, shear capacity, loss of stability, rupture, etc. The serviceability limit states are related to cracking, deflection, and vibration. Analysis of selected bridges and idealized structures without any over design was performed and the level of safety in the existing bridges was calculated. It was observed that most of the structures are over designed for serviceability and ultimate limit states. A study on existing bridges designed using Standard Specifications was carried out and a target safety index was then proposed by Nowak et al. (1996).

The analysis of a number of design cases indicates that unlike other structures, prestressed concrete girders are typically not governed by ultimate limit state. The number of prestressing strands is generally governed by the allowable tension stress at the final load stage. The ultimate limit states and the corresponding reliability indices represent component reliability rather than system, as observed by Tabsh and Nowak (1991). The LRFD Specifications were developed using the target reliability index for a structural component as  $\beta_T = 3.5$ . Tabsh and Nowak (1991) proposed that the target reliability index for structural components be taken as  $\beta_T = 3.5$  and for structural system as  $\beta_T = 5.5$  for ultimate limit states and  $\beta_T = 1.0$  for serviceability limit states.

## **2.4 LOAD MODELS**

### **2.4.1 General**

The development of load and load combination models had an important role in the development of the reliability based LRFD Specifications. Extensive research studies by Nowak (1987, 1991, 1993c, 1993d 1995, 1999), and Kulicki et al. (1994) were

focused on the development of load models representative of the truck loads on highway bridges in the United States. Load models are based on available data from truck surveys, material tests, and component testing.

## **2.4.2 Dead Load Models**

The gravity loads due to self weight of the structural and nonstructural components of a bridge contributes to the dead load. Depending on the degree of variation, the dead load components are divided into four categories: weight of factory made components, weight of cast in place concrete members, weight of wearing surface and miscellaneous weights (railings, curbs, luminaries, signs, conduits, pipes etc.) each having different bias factor (ratio of mean to nominal values) and coefficient of variation. Nowak et al. (1999) calculated the bias factors and coefficients of variation for each dead load category, based on data from material and component test data, which is summarized in Table 2.1.

**Table 2.1. Statistical Parameters for Dead Load**

| <b>Component</b>      | <b>Bias Factor</b> | <b>Coefficient of Variation</b> |
|-----------------------|--------------------|---------------------------------|
| Factory made members  | 1.03               | 0.08                            |
| Cast-in-place members | 1.05               | 0.10                            |
| Asphalt               | 90 mm              | 0.25                            |
| Miscellaneous         | 1.03-1.05          | 0.08-0.10                       |

## **2.4.3 Live Load Models**

### *2.4.3.1 General*

Several studies have been undertaken to model the live load on United States (U.S.) highway bridges to reflect actual truck traffic in the coming years and its effects

on bridges as accurately as possible. The uncertainty in the live load model is caused by unpredictability of the future trends with regard to configuration of axles and weights. The NCHRP 12-33 project was developed to determine appropriate models for bridge live loads and its results were incorporated into the LRFD Specifications (Nowak 1999). Knowledge of the statistical models including distribution of loads, rate of occurrence, time variation, and correlation with other load components is needed to model the loads accurately. A 75 year extrapolation of the traffic on U.S. bridges was done. Moments and shears were then calculated for these loads and it have been found that the shears and moments caused by the heaviest vehicles range from 1.5 to 1.8 times the design moment provided by Standard Specifications. Various possible truck positions were considered with varying degrees of correlation between them in order to arrive at the maximum moments and shears due to actual traffic loading.

#### *2.4.3.2 Live Load Model*

A live load model for highway bridges was developed by Nowak et al. (1991) from the truck survey data and weigh in motion measurements carried out by different state departments of transportation, mostly from the former source. A procedure for the calculation of live load moments and shears for highway girder bridges was proposed by Nowak et al. (1991). In this formulation the load components are treated as random variables and load combinations of dead load, live load, and dynamic load were considered. The findings by Nowak et al. suggest that a single truck causes maximum moment and shear for single lane bridges with spans up to 100 ft. and two trucks following behind each other control for longer spans. For two lane bridges, the maximum values are obtained for two trucks side by side with fully correlated trucks.

Nowak (1995) calibrated the LRFD Specifications using a probability-based approach. About 200 bridges were selected in this study, and for each bridge, load effects and load carrying capacities were calculated for various components were evaluated. Live load models were developed using weigh in motion (WIM) data that included the effects of presence of multiple trucks on the bridge in one and in adjacent

lanes. A reduction factor for multilane bridges was also calculated for wider bridges. Numerical models were developed for simulation of dynamic bridge behavior for single trucks and two trucks, side by side, due to inadequate field data.

Five candidate live loads identified for development of live load model for LRFD Specifications were as follows: (1) a single vehicle weighing a total of 57 tons, (2) a “family” of three loads consisting of tandem, a four axle single unit, with a tandem rear combination, and a 3-S-3 axle configuration taken together with a uniform load, preceding and following the above load group, (3) a slightly different combination of HS vehicle and the uniform load, involving HS-25 load followed and proceeded by a uniform load of 480 pounds per running foot of lane, with the uniformly distributed load broken for the HS vehicle, (4) a design “family” called HL-93 consisting of a combination of a design tandem, the HS-20 truck and a uniform load of 640 pounds per running foot of lane, (5) an equivalent uniform load in kips per foot of lane required to produce the same force effect as the envelope of exclusion vehicles for various span lengths. Considering the complex nature of this load case, it was eliminated.

For each of the four remaining configurations plots for center line moments for a simply supported beam, positive and negative moments at the  $0.4L$  point of a two-span continuous girder, with two equal spans, negative moment at the center pier, end shear and shear at both sides of interior support of a two span continuous girder were compared. Load model involving a combination of either a pair of 25- kip tandem axles and the uniform load, or the HS-20 and the uniform load, was found to produce the best fit to the exclusion vehicles and hence can be used as the design load in LRFD Specifications.

The parameters that influence the static component of live load are truck weight, axle loads, axle configuration, span length, position of vehicle on the bridge, number of vehicles on the bridge, stiffness of structural members and future growth. The bending moments for trucks are calculated and the cumulative distribution functions for simple spans 30 – 200 ft. is plotted on normal probability paper. Mean values of the moments are found to be about 0.7-0.85 of HS-20 moments and the slope of the cumulative

distribution functions (CDF's) gives the coefficient of variation which is about 0.2 – 0.35. Maximum moments came to about 1.4-1.8 of HS-20 moments. Mean shears were about 0.7-0.85 whereas maximum shears were 1.4-1.7 of HS-20 shears. The maximum moments and shears are then extrapolated to get 75 year maximum values. Their coefficient of variations can be then found by transformation of CDF's (i.e. by raising each CDF to a certain power depending on time, which gives the mean values after transformation). The slope of this transformed CDF gives the coefficient of variation  $V$ . Sometimes for a multiple truck occurrence there may be some degree of correlation between them. In the development of this model, the following assumptions were made:

- 1) Every 10<sup>th</sup> truck is followed by another truck with a headway distance less than 50ft.
- 2) Every 50<sup>th</sup> truck is followed by a partially correlated truck, and 3) every 100<sup>th</sup> truck is followed by fully correlated truck.

Various findings are indicated using figures and tables.

The factors on which the effect of live load depends are span length, truck weight, axle loads, axle configuration, and position of vehicle on the bridge, number of vehicles on the bridge, girder spacing and stiffness of structural members. The bending moments and shear force were calculated for each truck for a wide range of simple spans. And the cumulative distribution functions were plotted on the normal probability paper and extrapolations were made to find the maximum load effects for extended periods of time. The resulting bias factors were plotted for shear and moments. Then the girder distribution factors were calculated from finite element models.

Kulicki (1994) discussed the development of new models for bridge live load. National Transportation Research Board (TRB) in its study titled “Truck weight Limits – Issues and Options” reviewed different vehicle configurations allowed by various states, of which 22 configurations were chosen for study. The plot of bending moments in a simple span and two span continuous girders in the span range of 20 to 150 feet due to the chosen loading configurations and AASHTO's HS-20 truck loading showed that the HS-20 truck loading is not representative of the wide range of vehicles currently on U.S. highways.

#### **2.4.4 Dynamic Load Models**

Hwang et al. (1991) presented a dynamic load model for bridges in the U.S. based on simulations and consideration of field effects to find the statistical parameters for the dynamic load effect. An equivalent static load effect was considered for the dynamic load effect. The factors affecting the dynamic load are road surface roughness, bridge dynamics, and vehicle dynamics. Modal equations for bridges were modeled using analytical methods. The dynamic load allowance for the bridges was calculated using different truck types. The mean dynamic load was determined to be equal to 0.10 and 0.15 of the mean live load for one truck and two trucks, respectively. However, the dynamic load is specified as 0.33 of the live load in the LRFD Specifications.

#### **2.4.5 Joint Effect of Dead, Live and Dynamic Loads**

Nowak (1993b) modeled the joint effect of dead, live, and dynamic loads by considering the maximum 75-year combination of these loads using their individual statistical parameters. The live load was assumed to be a product of static live load and live load analysis factor  $P$ , having mean value of 1.0 and coefficient of variation of 0.12. The statistical parameters of the combination of dead load, live load and dynamic load depend on various factors such as span length, and number of lanes. For single lane the coefficient of variation was found to be 0.19 for most of the spans and 0.205 for very short spans. For two lane bridges, the coefficient of variation was found to be 0.18 for most spans and 0.19 for very short spans.

#### **2.4.6 Earthquake Load Model**

Earthquake loading is the most challenging load type to model owing to its high uncertainty and variation with time. Earthquake load can be represented as a function of ground acceleration, which is highly site specific, along with parameters specific to the structural system and structural component. Earthquake loading is presented by Nowak et al. (1999) as a product of three variables representing variation in ground acceleration, uncertainty in transition from load (ground acceleration) to load effect in a component



(moment, shear and axial forces), and uncertainty due to approximations in structural analysis. The AASHTO LRFD Specifications present the design values of the return period for an earthquake and its magnitude in the form of contour maps, based on probabilistic analysis.

The AASHTO Standard Specifications (1996) present the earthquake load as a function of the acceleration coefficient as obtained from contour maps, site effect coefficients that approximate the effect the soil profile type and importance classification allotted to all bridges having an acceleration coefficient greater than 0.29 for seismic performance categorization. The LRFD Specifications specifies the earthquake load in a similar manner as that of the Standard Specifications, but it introduces three categories of importance: critical bridges, essential bridges, and other bridges that are used to modify the load and resistance factors. The return period is assumed to be 475 years for essential and other bridges and 2500 years for critical bridges.

#### **2.4.7 Scour Effect Model**

Scour, although not considered as a load, can cause a significant effect on bridge performance due to load distribution, and is a major cause of bridge failure in the U.S. (Nowak 1999). Scour can be considered as an extreme event in bridge design. The three types of scour are long term channel degradation referring to scour across the entire waterway breadth, contraction scour referring to scour caused due to the constriction of the stream caused by bridge approach embankments, and local scour which refers to severe erosion around piers and abutments. Local and contraction scour generally occurs under the bridge and usually gets filled after flood events. The current AASHTO specifications do not specify how to consider scour effects in combination with various loading conditions. However, the approach to design is presented in the FHWA publication Stream Stability at Highway Structures and further work to evaluate scour effects is in progress (Nowak 1999).

#### **2.4.8 Vessel Collision Model**

Vessel collision is another extreme load which is very difficult to model due to its time varying effects. Time varying product of three variables representing variation in the vessel collision force, variation due to transition from vessel collision to load effect in a component, and variation due to approximations in structural analysis can be used to statistically represent the vessel collision effect. Vessel impact force depends on type, displacement tonnage and speed of vessels and other site specific factors such as waterway characteristics and geometry, vessel and/or barge configurations, and bridge type and geometry. Any one of the three different procedures to determine the vessel collision force provided in the AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges can be used. The LRFD Specifications uses different return periods with different importance classifications with three levels of statistical complexity. Vessel collision force is based on a return period of 1000 years for essential and other bridges, whereas for critical bridges the return period is 10,000 years (Nowak 1999).

#### **2.4.9 Load Combination Models**

Load combination is a random variable that can be represented by a probability distribution function (PDF) for statistical analysis. The load combination is the effect of simultaneous occurrence of two or more load components. The PDFs for critical load combinations should be generated and calibration performed to achieve a consistent risk level, but this is not possible in most cases due to numerical difficulties. Reliability analysis is the best alternative to find the critical load combination, where load and resistance factors are found such that the reliability of the structure is at a predefined target safety level.

The design values for load combinations in the AASHTO Standard Specifications are based on engineering judgment and past experience whereas the design values for factored load combinations present in the LRFD Specifications are based on statistical approach to attain a uniform reliability index of 3.5. The LRFD

Specifications are calibrated for basic load combinations only, due to the lack of a statistical database of correlation of extreme load events. Nowak et al. (1999) recommends a full probability based calibration of all loading events before choosing the critical load combination.

#### 2.4.10 Load Factors

Nowak (1999) recommended load factors which when used with specified resistance factors yield uniform safety for bridges, close to the target reliability index. For the dead loads due to factory made members and cast in place members the load factor was 1.25. For asphalt wearing surface weight, the load factor was calculated as 1.5 and the negative dead load can be obtained by multiplying the dead load by 0.85-0.90. The live load factor was given as 1.6 for Average Daily Truck Traffic (ADTT) =1000, and for ADTT=5000, the load factor is calculated as 1.70. The following combinations are suggested by Nowak (1999).

$$1.25 D + 1.50 D_A + 1.70(L+I) \quad (2.1)$$

$$1.25 D + 1.50 D_A + 1.40 W \quad (2.2)$$

$$-0.85 D - 0.50 D_A + 1.40 W \quad (2.3)$$

$$1.25 D + 1.50 D_A + 1.35(L+I) + 0.45 W \quad (2.4)$$

$$1.25 D + 1.50 D_A + \gamma_L (L+I) + 1.00 E \quad (2.5)$$

where:

$D$  = Dead load of structural components and non-structural attachments

$D_A$  = Dead load of asphalt wearing surface

$L$  = Live load

$I$  = Dynamic load

$W$  = Wind load

$E$  = Earthquake load

$\gamma_L$  = 0.25-0.50 for ADTT = 5000

- = 0.10-0.20 for ADTT = 1000
- = 0 for ADTT = 100

## **2.5 RESISTANCE MODELS**

### **2.5.1 General**

The development of the resistance parameters was critical to the development of the AASHTO LRFD Specifications. Nowak et al. (1994) studied the bending and shear resistances provided by reinforced and prestressed concrete bridge girders. For the development of the probability-based specifications, accurate prediction of the load carrying capacity of structural components is necessary, and the calculation of the difference between the mean capacity and nominal capacity is important.

### **2.5.2 Development of Resistance Models**

In the development of resistance models, the resistance was treated as a random variable due to the uncertainties involved in the material properties, dimensions, and methods of analysis. The material uncertainty is generally caused due to the variation in the strength of material, modulus of elasticity, cracking stress, and/or chemical composition. The variation in geometry, dimensions and/or section modulus causes fabrication uncertainty. Finally, the approximations in analysis and idealization of stress and strain curves induce analysis uncertainty. Given so many uncertainties and variations, a large database of material tests and component tests is required to accurately model the resistance. In absence of this database, analytical simulations were used by Nowak et al. (1994) for the analysis of large bridge components.

The bias factors (ratio of mean-to-nominal-capacity) were found for AASHTO prestressed concrete girders considering bending moment and shear capacities. The statistical properties for concrete, reinforcing steel, and prestressing steel were determined using available data. Monte Carlo technique was used to simulate moment-curvature curves corresponding to spans of 40, 60, and 80 ft. for AASHTO Type II, III, and IV prestressed concrete girders. The parameters used for AASHTO type IV girder

are as follows span = 80 ft., effective depth of prestressing steel,  $d_{ps} = 59$  in., area of prestressing steel,  $A_{ps} = 4.59$  in<sup>2</sup>. The results were as follows nominal moment = 5592 k-ft., bias factor = 1.033, coefficient of variation = 0.033.

The shear capacity was modeled using modified compression field theory. The parameters in this case were span = 80 ft., spacing of stirrups = 16 in. and area of steel  $A_v = 0.22$  in<sup>2</sup> which resulted in a nominal shear capacity of 219.2 kips with a bias factor of 1.067 and coefficient of variation of 0.0805. The shear resistance was calculated using Monte Carlo simulation. The overall results show that for moment capacities the bias factors for fabrication and materials was 1.04, for analysis 1.01, and for overall resistance 1.05. The corresponding coefficients of variation were found to be 0.04, 0.06, and 0.075. In the case of shear, the bias factors for fabrication and materials was 1.07, for analysis 1.075, and for overall resistance 1.15. The corresponding coefficients of variation were found to be 0.10, 0.10, and 0.14 (Nowak 1999).

### **2.5.3 Resistance Factors**

Nowak (1994) in his study found the resistance factors for prestressed concrete girders. The load factors from the LRFD Specifications were used and the target reliability index was set to 3.5. Using trial and error, resistance factors ( $\phi$ ) were calculated. The resistance factor for prestressed concrete girders was determined to be 1.00 for moment and 0.85 for shear. These resistance factors when used in conjunction with the LRFD specified load factors yield a uniform safety level for a wide range of span lengths.

## **2.6 LOAD DISTRIBUTION FACTORS**

### **2.6.1 General**

One of the major changes encountered by bridge engineers in the LRFD Specifications is in the load distribution factors (DFs), which are based on a detailed parametric study and recommendations by Zokaie et al. (1991). Many studies have been carried on later to verify the results obtained by Zokaie et al. and several

recommendations were given. These studies have shown that use of refined analysis instead of the approximate formulas recommended by Zokaie et al. (1991) yields accurate results, while the approximate results are generally conservative Barr et al. (2001), Chen et al. (1996), Ali et al (2003). While going through the available literature it was observed that researchers have not come to a universal conclusion about the effect of the presence of end and intermediate diaphragms on distribution factors. The distribution factors recommended by the LRFD Specifications are thought to be more accurate than those presented in the Standard Specifications, but still lack in accuracy, because the effect of various components of a typical bridge are not included, as indicated by Barr et al. (2001) and Chen et al. (1996).

### **2.6.2 Differences Between Standard and LRFD Load DFs**

The AASHTO Standard Specifications gives very simple expression for live load distribution factor for the girder bridges in  $S/D$  format, where  $D = 5.5$  for a bridge constructed with a concrete deck on prestressed concrete girders carrying two or more lanes of traffic and  $S$  is the girder spacing in feet. The effects of various parameters such as skew, continuity, and deck stiffness were ignored in this expression and it was found to be accurate for a few selected bridge geometries and was inaccurate once the geometry was changed, hence a need for development of a formula which holds good for a broad range of beam and slab bridges, including prestressed concrete bridges was felt. Much research was carried out to arrive at an accurate expression for DFs, using finite element analysis, grillage analysis and field tests. The DFs proposed by LRFD considers the effects of different parameters such as skew, deck stiffness, and span length. The LRFD Specifications provide correction factors for skewed bridges to be applied to distribution factors.

### **2.6.3 Development of Load Distribution Factors**

Zokaie et al. (1991) conducted a parametric study using detailed finite element models taking into account different parameters such as skew angle, girder spacing,

girder stiffness, and slab stiffness to calibrate the effect of these parameters on the girder distribution factors, which were ignored in the Standard Specifications formula. The finite element models used in the study were validated by data from hundreds of field tests from various DOTs across the U.S. from a variety of bridges. A computer program, GENDEK5A, was selected for the development of finite element models because this program can model the bridge system more accurately and generated good results as compared with field test results from many prototype bridges, as compared to other finite element programs.

Each finite element model for a bridge was generated and the design trucks were positioned in all possible ways so as to produce the maximum moments and shears, and then the distribution factors were calculated for each girder. This process was repeated for several hundreds of models for bridges across the nation at random and results were compared with actual field data. The database of results was then used to identify the key parameters affecting the distribution factors for a given bridge, and the correlation between each of them was determined by plotting them against one another (Zokaie 2000). The results from the study show that the parameters are not correlated. A model named “Average Bridge” was developed using the mean values of all the parameters except the one under consideration, which was varied to recognize its affects on the distribution factors under HS-20 truck loading. After significant testing Zokaie (2000) determined that girder spacing, span length, girder stiffness, and slab thickness control the distribution factors for a given bridge. The effects of these parameters were studied and Zokaie (2000) arrived at a simplified base formula provided in Figure 2.2, that represents the effects of different parameters on the distribution factors for girders and yields conservative results.

Further extensions to this base formulas were made by Zokaie (2000) to account for various factors such as presence of edge girder, continuity, and skew effect and correction factors to adjust the base formulas were proposed. The formulas that appear in the LRFD Specifications are slightly different from those developed from this study. Changes were made to account for the new live load required in the LRFD

Specifications which is different from the HS-20 trucks used for this study. A simple program called LDFAC (Zokaie et al. 1993) was developed to assist engineers in finding out the applicable distribution factors for a given bridge. Zokaie et al. (1991) recommends the use of accurate analysis if the geometry of the bridge is different from those considered in the study and a set of recommendations for such analysis is also given.

**TABLE 7. AASHTO-LRFD Formulas for Moment/Shear Distribution ( $g$ ) to Interior Girders**

| Bridge type<br>(1)                    | Bridge designed for one<br>traffic lane<br>(2)  | Bridge designed for two or more<br>traffic lanes<br>(3)  | Range of applicability<br>(4)  |
|---------------------------------------|---|--|--|
| (a) Moment                            |   |  |  |
| Beam and slab                         | $0.12 + 2 \left( \frac{S}{14f} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_t}{L t_s^3} \right)^{0.1}$ | $0.15 + \left( \frac{S}{9.5f} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_t}{L t_s^3} \right)^{0.1}$                         | $3.5f \leq S \leq 16f$<br>$20f \leq L \leq 240f$<br>$N_b \geq 4$   |
| Concrete box girders                  | $\left( 3.5 + \frac{S}{1.8f} \right) \left( \frac{f}{L} \right)^{0.35} \left( \frac{1}{N_c} \right)^{0.45}$             | $2 \left( \frac{13}{N_c} \right)^{0.3} + \left( \frac{S}{5.8f} \right) \left( \frac{f}{L} \right)^{0.25}$                                      | $7f \leq S \leq 13f$<br>$60f \leq L \leq 240f$<br>$3 \leq N_c$<br>if $N_c > 8$ use $N_c = 8$   |
| Slab                                  | $\frac{2f + \sqrt{L_1 W_1}}{4}$   | $3.5f + 0.06\sqrt{L_1 W_1}$  | $8f \leq L \leq 70f$<br>$12f \leq W \leq 100f$   |
| Spread box beams                      | $2 \left( \frac{S}{3f} \right)^{0.35} \left[ \left( \frac{S}{L} \right) \left( \frac{d}{L} \right) \right]^{0.25}$      | $2 \left( \frac{S}{6.3f} \right)^{0.6} \left[ \left( \frac{S}{L} \right) \left( \frac{d}{L} \right) \right]^{0.125}$                           | $6f \leq S \leq 11.5f$<br>$20f \leq L \leq 140f$<br>$1.5f \leq d \leq 5.4f$<br>$N_b \geq 3$  |
| Multibox beam decks                   | $1.2k \left( \frac{b}{L} \right)^{0.5} \left( \frac{I}{J} \right)^{0.25}$<br>$k = 2.5(N_b)^{-0.2} \geq 1.5$             | $2k \left( \frac{b}{25.4f} \right)^{0.6} \left( \frac{b}{L} \right)^{0.2} \left( \frac{I}{J} \right)^{0.06}$<br>$k = 2.5(N_b)^{-0.2} \geq 1.5$ | $3f \leq b \leq 5f$<br>$20f \leq L \leq 120f$<br>$2.92f \leq b \leq 5f$<br>$5 \leq N_b \leq 20$  |
| (b) Shear                             |   |  |  |
| Beam and slab (slab on girder)        | $2 \left( 0.36 + \frac{S}{25f} \right)$   | $0.4 + \frac{S}{6f} - 2 \left( \frac{S}{35f} \right)^{2.0}$  | $3.5f \leq S \leq 16f$<br>$20f \leq L \leq 200f$<br>$0.375f \leq t_s \leq 12.0f$<br>$0.48f^4 \leq K_t \leq 337f$<br>$N_b \geq 4$       |
| Concrete box girder                   | $2 \left( \frac{S}{9.5f} \right)^{0.6} \left( \frac{d}{L} \right)^{0.1}$  | $2 \left( \frac{S}{7.3f} \right)^{0.9} \left( \frac{d}{L} \right)^{0.1}$   | $7f \leq S \leq 13f$<br>$60f \leq L \leq 240f$<br>$3 \leq N_c$   |
| Spread box beams                      | $2 \left( \frac{S}{10f} \right)^{0.6} \left( \frac{d}{L} \right)^{0.1}$<br>for $S \geq 11.5'$ use lever rule            | $2 \left( \frac{S}{7.4f} \right)^{0.8} \left( \frac{d}{L} \right)^{0.1}$   | $6f \leq S \leq 11.5f$<br>$20f \leq L \leq 135.5f$<br>$N_b \geq 3$   |
| Multibox beams                        | $2 \left( \frac{S}{10.83L} \right)^{0.15} \left( \frac{I}{J} \right)^{0.05}$  | $2 \left( \frac{b}{13f} \right)^{0.4} \left( \frac{b}{L} \right)^{0.1} \left( \frac{I}{J} \right)^{0.05}$                                      | $3f \leq b \leq 5f$<br>$20f \leq L \leq 105f$<br>$5 \leq N_b \leq 20$<br>$1.2f^4 \leq J \leq 29.5f^4$<br>$1.92f^4 \leq I \leq 29.5f^4$ |
| Note: $f = 304.8 \text{ mm (1.0 ft)}$ |   |  |  |

**Figure 2.2. Distribution Factors Proposed by Zokaie (Zokaie 2000)**



#### 2.6.4 Evaluation of LRFD Load DFs

A number of studies have been carried out to evaluate the results obtained by Zokaie et al (1991). The methods of analysis used and recommendations made by them are briefly documented as follows.

Barr et al. (2001) determined the effects of different components of a typical girder bridge and proposed a representative expression for the live load distribution factors. The effects of lifts, intermediate diaphragms, end diaphragms, continuity, skew angle, and load type on the live load distribution factors for prestressed concrete girder bridges were considered. A finite element model of a three-span prestressed concrete bridge girder was developed and compared with the field static live load test values for accuracy. Test data from the SR18/SR516 bridge, designed by the Washington DOT having three spans with lengths of 80, 137 and 80 ft. and a skew angle of  $40^0$ , was used by Barr et al. (2001) to validate the analytical model. The model was then loaded with an HS-20 truck load and more evaluation was done using several locations of the truck and many variations in the geometry including addition of lifts and continuity. The moments calculated using the analytical models were slightly larger than the measured moments in all cases with the largest discrepancy of 6 percent. Then many variations were made to this model to evaluate the effects of different parameters on the load distribution factors. The results show that the LRFD distribution factors are always conservative and sometimes over conservative (Barr et al. 2001).

A study conducted by Ali et al. (2003) also confirmed the conservatism of the LRFD distribution factors. Ali et al. (2003) evaluated the effects due to skew of bridge and addition of transverse diaphragms on the load distribution factors using finite-element models. The parameters of the study were girder spacing (1.8 – 2.7 m), span length (25 – 35 m), skew angle ( $0 - 60^0$ ), and different arrangements of internal transverse diaphragms. Bridge models with three spans of 25, 30 and 35 m, varying the girder spacing as 1.8, 2.4 and 2.7 m were considered. The models were loaded with HS-20 truck according to the AASHTO Standard Specifications. The results of the study indicated that the skew has the greatest effect on the load distribution factor. The load

distribution factors for skew bridges were found to be always less than that of right bridges (with no skew). In all the cases the load distribution factors of the LRFD Specifications were found to be conservative and in some cases over conservative.

In a study by Schwarz et al. (2001) the DFs were measured for vehicles with a variety of axle spacings, number of axles, and Gross Vehicle Weights (GVWs). The measured DFs were compared to the ones in both the LRFD and Standard Specifications and also those obtained from grillage analysis. For a given bridge, the one lane DFs were found to be the largest in the girders beneath the loaded lanes. The LRFD and Standard DFs for one lane were found to be conservative by 17 percent. For shorter spans, Schwarz et al. (2001) propose that neglecting diaphragms can improve the accuracy of the model. Two lane DFs were obtained by side-by-side truck positioning and superimposing average one-lane DFs. The AASHTO Standard and LRFD DFs were found to be conservative for two lane case also.

Chen et al. (1996) investigated the application of the load DFs proposed by Zokaie et al. (1991) to modern prestressed concrete bridges made of I-girders and spread box girders with larger span-to-depth ratios using refined analysis methods and recommended changes to the proposed DFs. The authors indicated that the average I-beam span length of 48 ft. considered by Zokaie et al. (1991) for arriving at the DFs is rather short for I-beam bridges, which are more likely to be 80 to 90 ft. long. The finite element analysis showed the DFs given by the LRFD Specifications to be over conservative by at least 18 percent for interior girders and 4-12 percent for exterior girders, and if it is used instead of LRFD DFs reduces the required release strength of concrete or allows 4-5% increase in span length for the same section. The effect of diaphragms is also ignored in the development of the LRFD DFs which add to their over conservatism (Chen et al.). All the studies supported the use of finite element method for the analysis of bridge system (Zellin et al. (1976), Zokaie et al. (1991), LRFD Bridge Specifications (1994) etc).

Nowak (1999) determined the girder distribution factors from finite element models based on linear behavior of girders and slab. Spans ranging from 30 to 200 ft.

were taken into account for five different girder spacing of 4, 6, 8, 10 and 12 ft. Nowak found that GDF's proposed by AASHTO Standard specifications are conservative for larger girder spacing and less than calculated for shorter spans and girder spacing, and were in good agreement with the Zokaie's results.

Another study by Puckett (2001) compared the results obtained by Zokaie et al (1991) by an independent finite strip method analysis of a slab on Girder Bridge. Puckett validated the results of Zokaie et al. only for interior beams. The largest discrepancy between the two was found to be 7%. Under the situation where different studies have not come to a conclusion as to the over conservatism of LRFD distribution factors, the use of accurate analysis is the best option, as indicated by all the studies.

## **2.6.5 Effect of Various Parameters**

### *2.6.5.1 General*

A study by Barr et al. (2001) indicated that Zokaie et al. (1991) did not considered the effect of lifts in their study and included the effect of diaphragms in pilot study but not in main study and the factor proposed by them for continuity was not included in the LRFD specifications. As a result the distribution factor proposed in LRFD specifications still do not consider the effects of the components of a typical bridge and are based on the results of analysis using HS-20 loading of simply supported bridges but are more accurate than the ones proposed in the *AASHTO Standard Specifications* (1996). Chen and Aswad (1996) in their study found the LRFD distribution factors to be uneconomically conservative. The effect of different parameters as indicated by different studies has been summarized as follows

### *2.6.5.2 Bridge Skew*

The skew of the bridge had little effect on distribution factor for small angles, and reduction in live-load distribution factor was observed for larger angles. The LRFD expression for skew factor was found to be appropriate for the behavior observed by different studies.

#### *2.6.5.3 Intermediate and End Diaphragms.*

The addition of end diaphragms was found to decrease the distribution factor for exterior and interior girders and the reduction was in the range of 6% to 25% for various skew angles. The presence of intermediate diaphragms slightly increased the live load distribution factor at low skew angles but at higher skew angles ( $\geq 30^\circ$ ) they proved to be slightly beneficial.

#### *2.6.5.4 Continuity*

The continuity increased the distribution factor regardless of the skew for exterior girders and it reduced the distribution factors for interior girders for low skew angles but increased the distribution factor for greater skew angles.

#### *2.6.5.5 Span Length*

The increase in span length was found to increase the load distribution factor for the external girders while this had little effect on the internal girders.

#### *2.6.5.6 Lifts*

The results of the study by Barr et al. indicates that the addition of lift reduced the distribution factor by 17% for exterior girder and by 11% for interior girder which can be explained by the fact that the presence of lifts slightly increases the composite girder stiffness.

#### *2.6.5.7 Other Parameters*

As expected the increase in girder spacing increased the load distribution factor. The LRFD specifications recommends the use of truck load plus lane load but the distribution factors were developed using the truck loading only and the same was applied to the lane loading. But, it was found from the study by Barr et al (2001) that the distribution factor for lane loading is always smaller as compared to the truck loading which if considered can increase the economy of the design.

Barr et al. (2001) proposed that if the effects of different parameters have been considered in the design of SR18/SR516 Bridge the required release strength of the girder concrete could have been reduced from 7400 psi to 6400 psi or the bridge could have been designed for a 39% higher live load.

## **2.7 IMPACT OF AASHTO LRFD SPECIFICATIONS ON SHEAR DESIGN**

### **2.7.1 General**

A number of studies have been carried on to assess the impact of LRFD Specifications on bridge design. A study by Shahawy et al. indicates that from shear considerations AASHTO Standard specifications (1989) is superior to LRFD (1994). Detailed studies by Zokaie et al. (2003) and Richard et al. (2002) suggests that LRFD design is more conservative and requires higher prestress or reinforcement as compared to the design by Standard Specifications, due to various factors as described by them.

### **2.7.2 Shear Design of Prestressed Concrete Girders**

Shahawy et al. (1996) compared the shear provisions in the AASHTO Standard Specifications (1989) and LRFD Specifications using laboratory tests on AASHTO Type II prestressed concrete girders. The AASHTO Standard Specifications are based on constant 45-degree truss analogy for shear, whereas LRFD adopts variable truss analogy based on modified compression field theory for its shear provisions. As a part of laboratory testing 20 full-scale prestressed concrete girders were used with variable span, amount of shear reinforcement, shear span and strand diameter. Three of the girders were tested without any shear reinforcement in order to figure out the contribution of concrete to shear strength,  $V_c$ .

Shahawy et al. (1996) found that the AASHTO Standard Specifications gives a good estimate of the shear strength of the girders and is conservative regardless of the shear reinforcement ratio, whereas the LRFD Specifications overestimates the shear strength of girders having high reinforcement ratios. The shear provisions of AASHTO Standard Specifications were found to agree with the test results in almost all the cases,

whereas for  $a/d$  ratios less than 1.5, LRFD (1994) overestimates the shear strength and for  $a/d$  more than 2.0 LRFD underestimates the shear strength. The predictions of AASHTO Standard Specifications for  $V_c$  are also found to be better than that of LRFD, both being conservative as compared to test results. The overall results for shear indicate the superiority of AASHTO Standard Specifications (1989) over LRFD Specifications (1994).

### 2.7.3 AASHTO Type III Girder Bridge

Richard et al. (2002) compared the design of AASHTO Type III Girder Bridge using the *AASHTO Standard Specifications for Bridges*, 16th Edition, and the *AASHTO LRFD Bridge Design Specifications*. The authors found the bridge design to be same in most respects irrespective of the Specifications used. The most significant changes observed by them were in the shear design where the skew factor and reinforcement requirements in LRFD Specifications required increased concrete strength and reinforcement. An increase in reinforcement in deck overhang and in wing wall was also observed by the authors, due to increased collision force. The design of bridges using LRFD specifications was found to be more calculation-intensive and complex. The design experience and conclusions were limited to a single span AASHTO Type III girder bridges.

The LRFD Specifications allows the distribution of permanent loads of and on the deck to be distributed uniformly among the beams and/or stringers (LRFD Specifications Article 4.6.2.2.1) which is a significant change from the Standard Specifications design practice where the dead loads due to parapets, sidewalks, railings were applied only to the exterior girder. An increase in non-composite dead load by 9% and decrease in composite dead load by 50% on the exterior girder, while a decrease in non-composite dead load by 4% and an increase in composite dead load by 97% on the interior girder were observed when LRFD Specifications is followed, as compared to the design by Standard Specifications (Richard et al. 2002). The Standard Specifications required the bridge to be designed for HS-25 loading, which is 125% of the AASHTO

HS-20 truck load or a design lane load comprising of 800 plf distributed load plus 22.5 kip or 32.5 kip point load for flexure or shear design cases, respectively. The LRFD Specifications adopts HL-93 load case for bridge design, which consists of a 36 ton design truck or design tandem and a 640 plf design lane load. The shear and bending moment after load distribution for both load cases were found to be roughly comparable.

Richard et al. (2002) found that LRFD design requires same number of prestressing strands as that of standard Specifications design but a higher concrete strength was required which could be explained as an effect of changes in live loads, load distribution factors, impact factors, skew factors and prestressing losses. The LRFD design effected the shear design significantly as the requirement of shear reinforcement went up substantially which was a result of increase in live load distribution factor for shear and a constant skew factor. LRFD design of the overhang is significantly different from that of standard design and it requires more reinforcement.

#### **2.7.4 Post-Tensioned Girder Bridges**

Zokaie et al. (2003) reviewed the impact of LRFD specifications on the design post tensioned concrete box girder bridges and highlighted the changes in the Specifications which lead to the requirement of higher post tensioning. Although the present study deals with prestressed concrete girder bridges, it may be of interest to find out the cause of the requirement of higher post tensioning which also may cause an increase in required prestress force.

The change in design live load was found to be one of the factors. The “Dual Truck” loading in LRFD Specifications increases the negative moment at interior supports which require additional negative reinforcement. The major changes in the load distribution factors were another factor which influenced the design. The load factors for different limit states are different in LRFD Specifications as compared to the fixed load factors in Standard Specifications however the allowable stresses are almost same in both the Specifications. The prestress loss equations are slightly changed in the LRFD Specifications and are more conservative as compared to Standard ones. Zokaie et al.

(2003) carried on detailed design for two different cases and found that self weight is nearly the same irrespective of the Specifications used. The live load response in LRFD case was much higher than LFD. The impact factor was higher but the load distribution factor for moment went down for LRFD design case. Service limit state-III which checks the tensile stresses in bottom fiber governed in both the cases and required 13% additional post tensioning for LRFD based design. Zokaie et al. (2003) did not considered shear in their design.

## **2.8 RESEARCH NEEDS**

All of these findings in previous studies are limited to the bridges considered in the design, and may vary significantly by changing the bridge geometry, girder spacing and other parameters. There is a need for a more rigorous study to find the effect of LRFD Specifications on bridge design by changing various parameters such as span length, spacing between the girders etc. Also the prestressed concrete bridges typical to Texas including the AASHTO Type IV and Type C girder bridges have not been considered in any previous studies which encouraged the researchers to take them into account in the present study.



### **3. TXDOT BRIDGE DESIGN PRACTICES**

#### **3.1 INTRODUCTION**

Texas Department of Transportation (TxDOT) specifies design recommendations for bridge engineers provided in *TxDOT Bridge Design Manual* (TxDOT 2001). This manual is primarily aimed to bring consistency in the design of bridges in Texas. The manual gives specific recommendations for design where Standard Specifications gives options to the designers. The manual also includes simplifications for bridge design. The manual is based on *Standard Specifications for Highway Bridges* except for few sections which are based on previous studies and experiences. A evaluation of some of the simplified procedures given by TxDOT Bridge Design Manual (TxDOT 2001) is carried out in this study. The impact of these simplifications and their applicability while using LRFD Specifications is discussed below.

#### **3.2 MODULAR RATIO BETWEEN SLAB AND GIRDER CONCRETE**

##### **3.2.1 General**

The TxDOT design methodology for prestressed concrete bridges is to assume the concrete strengths at release and at service at the beginning of bridge girder design. Typically the concrete strength at release,  $f'_{ci}$  is taken as 4000 psi and the concrete strength at service,  $f'_c$  is assumed to be 5000 psi. The concrete strengths are optimized and selected during the design process. As the actual concrete strengths are not known at the beginning of design process, the modular ratio between the slab and girder concrete ( $n$ ) is chosen as unity. This modular ratio needs to be updated once the actual concrete strengths are selected. However, the TxDOT Bridge Manual allows for the use of modular ratio as unity throughout the design. The effect of haunch on the composite properties of the girder is not taken into account for bridges designed using TxDOT methodology. It is assumed that the haunch effect neutralizes the impact of the assumption of modular ratio being unity, and will not affect

the girder designs based on Standard Specifications significantly. This simplification is also followed by the TxDOTs Bridge Design Program PSTRS14 (TxDOT 2004).

The live load moment and shear distribution factors (DFs) specified by Standard Specifications do not depend on the modular ratio between slab and girder concrete. The live load moment DFs specified by the LRFD Specifications however involves a term  $K_g$ , which depends on the modular ratio. The assumption of modular ratio as unity thus needs to be evaluated for the design based on LRFD Specifications. The impact of assuming the modular ratio as unity is evaluated in this study, when designs are based on Standard and LRFD Specifications. However, the haunch effect is ignored as the actual dimensions of the haunch are not provided for this study. The evaluation of the impact of not updating the modular ratio is carried on for Type IV girder with skew of 0 degrees. The skew is not a factor for this evaluation as the modular ratio has no impact on skew correction factors. Similar trends are expected for Type C girders. The results for AASHTO Type IV girders are presented below.

### **3.2.2 Methodology**

The methodology discussed in Section 4 is used with slight modifications. The Matlab program used for the parametric study was modified for this purpose. The design is first carried out assuming a modular ratio of unity. Once the concrete strengths are obtained, the actual modular ratio is evaluated and the program is run using the actual modular ratio. The refined optimized concrete strengths are thus obtained. The modular ratio is again calculated using the refined concrete strengths. The program is run again until the difference in the modular ratios is less than 0.05. Once the modular ratio converges within this limit, the camber, and the flexure and shear design limit states are evaluated. The design results thus obtained are compared with the ones evaluated in the parametric study.

### 3.2.3 Impact on Live Load Moment and Shear DFs

The impact of not updating the modular ratio on the live load moment and shear DFs is evaluated in this section. The results are presented in Tables 3.1 and 3.2 for moment and shear DFs respectively. The live load moment and shear DFs specified by Standard Specifications do not depend on the modular ratio, hence no change is found. The live load moment DFs specified by the LRFD Specifications were found to be decreasing in the range of 1 percent to 3 percent. The live load shear DFs specified by the LRFD Specifications are not dependent on modular ratio, thus no difference was observed.

**Table 3.1. Comparison of Live Load Moment DFs  
(Skew = 0°, Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Standard         |             |                | LRFD             |             |                |
|----------------------|------------|------------------|-------------|----------------|------------------|-------------|----------------|
|                      |            | Updated <i>n</i> | TxDOT Meth. | Difference (%) | Updated <i>n</i> | TxDOT Meth. | Difference (%) |
| 6                    | 90         | 0.545            | 0.545       | 0.0            | 0.546            | 0.552       | -1.0           |
|                      | 100        | 0.545            | 0.545       | 0.0            | 0.532            | 0.537       | -1.0           |
|                      | 110        | 0.545            | 0.545       | 0.0            | 0.518            | 0.523       | -0.9           |
|                      | 120        | 0.545            | 0.545       | 0.0            | 0.503            | 0.512       | -1.7           |
|                      | 130        | 0.545            | 0.545       | 0.0            | 0.490            | 0.501       | -2.2           |
|                      | 133        | -                | -           | -              | 0.482            | 0.498       | -3.2           |
|                      | 136        | 0.545            | 0.545       | 0.0            | -                | -           | -              |
| 8                    | 90         | 0.727            | 0.727       | 0.0            | 0.668            | 0.675       | -1.0           |
|                      | 100        | 0.727            | 0.727       | 0.0            | 0.650            | 0.656       | -1.0           |
|                      | 110        | 0.727            | 0.727       | 0.0            | 0.629            | 0.639       | -1.7           |
|                      | 120        | 0.727            | 0.727       | 0.0            | 0.607            | 0.625       | -2.8           |
|                      | 124        | 0.727            | 0.727       | 0.0            | -                | -           | -              |
| 8.67                 | 90         | 0.788            | 0.788       | 0.0            | 0.708            | 0.715       | -1.0           |
|                      | 100        | 0.788            | 0.788       | 0.0            | 0.688            | 0.695       | -1.0           |
|                      | 110        | 0.788            | 0.788       | 0.0            | 0.665            | 0.677       | -1.8           |
|                      | 116        | -                | -           | -              | 0.652            | 0.667       | -2.3           |
|                      | 119        | 0.788            | 0.788       | 0.0            | -                | -           | -              |

**Table 3.2. Comparison of Live Load Shear DFs  
(Skew = 0°, Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Standard         |             |                | LRFD             |             |                |
|----------------------|------------|------------------|-------------|----------------|------------------|-------------|----------------|
|                      |            | Updated <i>n</i> | TxDOT Meth. | Difference (%) | Updated <i>n</i> | TxDOT Meth. | Difference (%) |
| 6                    | 90         | 0.545            | 0.545       | 0.0            | 0.671            | 0.671       | 0.0            |
|                      | 100        | 0.545            | 0.545       | 0.0            | 0.671            | 0.671       | 0.0            |
|                      | 110        | 0.545            | 0.545       | 0.0            | 0.671            | 0.671       | 0.0            |
|                      | 120        | 0.545            | 0.545       | 0.0            | 0.671            | 0.671       | 0.0            |
|                      | 130        | 0.545            | 0.545       | 0.0            | 0.671            | 0.671       | 0.0            |
|                      | 133        | -                | -           | -              | 0.671            | 0.671       | 0.0            |
|                      | 136        | 0.545            | 0.545       | 0.0            | -                | -           | -              |
| 8                    | 90         | 0.727            | 0.727       | 0.0            | 0.814            | 0.814       | 0.0            |
|                      | 100        | 0.727            | 0.727       | 0.0            | 0.814            | 0.814       | 0.0            |
|                      | 110        | 0.727            | 0.727       | 0.0            | 0.814            | 0.814       | 0.0            |
|                      | 120        | 0.727            | 0.727       | 0.0            | 0.814            | 0.814       | 0.0            |
|                      | 124        | 0.727            | 0.727       | 0.0            | -                | -           | -              |
| 8.67                 | 90         | 0.788            | 0.788       | 0.0            | 0.861            | 0.861       | 0.0            |
|                      | 100        | 0.788            | 0.788       | 0.0            | 0.861            | 0.861       | 0.0            |
|                      | 110        | 0.788            | 0.788       | 0.0            | 0.861            | 0.861       | 0.0            |
|                      | 116        | -                | -           | -              | 0.861            | 0.861       | 0.0            |
|                      | 119        | 0.788            | 0.788       | 0.0            | -                | -           | -              |

### 3.2.4 Impact on Distributed Live Load Moments and Shears

The impact of not updating the modular ratio on the distributed live load moments and shears is evaluated in this section. The results are presented in Tables 3.3 and 3.4 for moments and shears respectively. The live load moments and shears specified by Standard Specifications do not change as the DFs remain the same after updating the modular ratio. The live load moment DFs specified by the LRFD Specifications were found to be decreasing in the range of 1 percent to 3 percent. This is due to the change in the live load moment DFs. The live load shears specified by the LRFD Specifications decreased in the range of 3 percent to 6 percent. This change is

caused due to the change in the distance of critical section, which is obtained from the transverse shear design.

**Table 3.3. Comparison of Distributed Live Load Moments  
(Skew = 0°, Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Standard         |             |                | LRFD             |             |                |
|----------------------|------------|------------------|-------------|----------------|------------------|-------------|----------------|
|                      |            | Updated <i>n</i> | TxDOT Meth. | Difference (%) | Updated <i>n</i> | TxDOT Meth. | Difference (%) |
| 6                    | 90         | 885.4            | 885.4       | 0.0            | 1298.2           | 1310.7      | -1.0           |
|                      | 100        | 997.4            | 997.4       | 0.0            | 1469.6           | 1483.8      | -1.0           |
|                      | 110        | 1108.8           | 1108.8      | 0.0            | 1643.6           | 1659.4      | -1.0           |
|                      | 120        | 1219.1           | 1219.1      | 0.0            | 1807.2           | 1837.8      | -1.7           |
|                      | 130        | 1328.5           | 1328.5      | 0.0            | 1973.9           | 2019.3      | -2.2           |
|                      | 133        | -                | -           | -              | 2008.0           | 2074.3      | -3.2           |
|                      | 136        | 1393.7           | 1393.7      | 0.0            | -                | -           | -              |
| 8                    | 90         | 1180.5           | 1180.5      | 0.0            | 1588.0           | 1603.8      | -1.0           |
|                      | 100        | 1329.9           | 1329.9      | 0.0            | 1796.3           | 1814.1      | -1.0           |
|                      | 110        | 1478.4           | 1478.4      | 0.0            | 1992.9           | 2027.3      | -1.7           |
|                      | 120        | 1625.5           | 1625.5      | 0.0            | 2181.3           | 2243.7      | -2.8           |
|                      | 124        | 1683.9           | 1683.9      | 0.0            | -                | -           | -              |
| 8.67                 | 90         | 1279.3           | 1279.3      | 0.0            | 1681.7           | 1698.5      | -1.0           |
|                      | 100        | 1441.2           | 1441.2      | 0.0            | 1901.8           | 1920.9      | -1.0           |
|                      | 110        | 1602.2           | 1602.2      | 0.0            | 2107.0           | 2146.2      | -1.8           |
|                      | 116        | -                | -           | -              | 2230.2           | 2283.0      | -2.3           |
|                      | 119        | 1745.7           | 1745.7      | 0.0            | -                | -           | -              |

**Table 3.4. Comparison of Distributed Live Load Shears**  
**(Skew = 0°, Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Standard         |             |                | LRFD             |             |                |
|----------------------|------------|------------------|-------------|----------------|------------------|-------------|----------------|
|                      |            | Updated <i>n</i> | TxDOT Meth. | Difference (%) | Updated <i>n</i> | TxDOT Meth. | Difference (%) |
| 6                    | 90         | 41.9             | 41.9        | 0.0            | 69.3             | 73.5        | -5.8           |
|                      | 100        | 42.2             | 42.2        | 0.0            | 72.6             | 76.5        | -5.2           |
|                      | 110        | 42.4             | 42.4        | 0.0            | 75.7             | 79.4        | -4.6           |
|                      | 120        | 42.6             | 42.6        | 0.0            | 78.7             | 82.1        | -4.2           |
|                      | 130        | 42.7             | 42.7        | 0.0            | 81.4             | 84.8        | -3.9           |
|                      | 133        | -                | -           | -              | 82.8             | 85.5        | -3.2           |
|                      | 136        | 43.5             | 43.5        | 0.0            | -                | -           | -              |
| 8                    | 90         | 55.9             | 55.9        | 0.0            | 84.4             | 89.3        | -5.5           |
|                      | 100        | 56.3             | 56.3        | 0.0            | 88.5             | 92.9        | -4.8           |
|                      | 110        | 56.6             | 56.6        | 0.0            | 92.0             | 96.4        | -4.6           |
|                      | 120        | 56.8             | 56.8        | 0.0            | 95.8             | 99.7        | -3.9           |
|                      | 124        | 56.8             | 56.8        | 0.0            | -                | -           | -              |
| 8.67                 | 90         | 60.6             | 60.6        | 0.0            | 89.3             | 94.4        | -5.4           |
|                      | 100        | 61.0             | 61.0        | 0.0            | 93.7             | 98.3        | -4.7           |
|                      | 110        | 61.3             | 61.3        | 0.0            | 97.3             | 101.9       | -4.5           |
|                      | 116        | -                | -           | -              | 99.9             | 104.1       | -4.0           |
|                      | 119        | 61.5             | 61.5        | 0.0            | -                | -           | -              |

### 3.2.5 Impact on Required Number of Strands

The impact of not updating the modular ratio on the required number of strands is evaluated in this section. The results are presented in Table 3.5. The required number of strands is found to be increasing when the modular ratio is updated. The increase is negligible. The increase in the number of strands is a result of the changed composite properties.

**Table 3.5. Comparison of Required Number of Strands**  
**(Skew = 0°, Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Standard         |             |                | LRFD             |             |                |
|----------------------|------------|------------------|-------------|----------------|------------------|-------------|----------------|
|                      |            | Updated <i>n</i> | TxDOT Meth. | Difference (%) | Updated <i>n</i> | TxDOT Meth. | Difference (%) |
| 6                    | 90         | 24               | 24          | 0.0            | 26               | 26          | 0.0            |
|                      | 100        | 32               | 32          | 0.0            | 34               | 34          | 0.0            |
|                      | 110        | 38               | 38          | 0.0            | 42               | 42          | 0.0            |
|                      | 120        | 48               | 48          | 0.0            | 54               | 54          | 0.0            |
|                      | 130        | 62               | 60          | 3.3            | 68               | 68          | 0.0            |
|                      | 133        | -                | -           | -              | 78               | 74          | 5.4            |
|                      | 136        | 74               | 70          | 5.7            | -                | -           | -              |
| 8                    | 90         | 30               | 30          | 0.0            | 32               | 32          | 0.0            |
|                      | 100        | 40               | 40          | 0.0            | 42               | 42          | 0.0            |
|                      | 110        | 52               | 50          | 4.0            | 54               | 54          | 0.0            |
|                      | 120        | 66               | 64          | 3.1            | 70               | 70          | 0.0            |
|                      | 124        | 78               | 74          | 5.4            | -                | -           | -              |
| 8.67                 | 90         | 32               | 32          | 0.0            | 34               | 34          | 0.0            |
|                      | 100        | 42               | 42          | 0.0            | 44               | 44          | 0.0            |
|                      | 110        | 54               | 54          | 0.0            | 58               | 58          | 0.0            |
|                      | 116        | -                | -           | -              | 68               | 68          | 0.0            |
|                      | 119        | 70               | 70          | 0.0            | -                | -           | -              |

### 3.2.6 Impact on Required Concrete Strengths

The impact of not updating the modular ratio on the required concrete strengths at release and at service is evaluated in this section. The results are presented in Tables 3.6. and 3.7. The required concrete strengths at release and at service are found to be increasing in few cases when the modular ratio is updated. However, the increase is negligible. The increase in the required concrete strengths is due to the increase in the number of strands, which increases the stresses in the girder, subsequently requiring higher concrete strengths.

**Table 3.6. Comparison of Required Concrete Strength at Release**  
**(Skew = 0°, Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Standard         |             |                | LRFD             |             |                |
|----------------------|------------|------------------|-------------|----------------|------------------|-------------|----------------|
|                      |            | Updated <i>n</i> | TxDOT Meth. | Difference (%) | Updated <i>n</i> | TxDOT Meth. | Difference (%) |
| 6                    | 90         | 4000.0           | 4000.0      | 0.0            | 4000.0           | 4000.0      | 0.0            |
|                      | 100        | 4000.0           | 4000.0      | 0.0            | 4009.7           | 4009.7      | 0.0            |
|                      | 110        | 4244.3           | 4244.3      | 0.0            | 4739.7           | 4739.7      | 0.0            |
|                      | 120        | 5246.7           | 5246.7      | 0.0            | 5930.6           | 5930.6      | 0.0            |
|                      | 130        | 6368.1           | 6403.0      | -0.5           | 6510.0           | 6510.0      | 0.0            |
|                      | 133        | -                | -           | -              | 6843.2           | 6655.0      | 2.8            |
|                      | 136        | 6725.0           | 6613.4      | 1.7            | -                | -           | -              |
| 8                    | 90         | 4000.0           | 4000.0      | 0.0            | 4000.0           | 4000.0      | 0.0            |
|                      | 100        | 4478.2           | 4478.3      | 0.0            | 4707.7           | 4707.7      | 0.0            |
|                      | 110        | 5698.6           | 5456.6      | 4.4            | 5893.2           | 5893.2      | 0.0            |
|                      | 120        | 6524.3           | 6538.4      | -0.2           | 6582.9           | 6582.9      | 0.0            |
|                      | 124        | 6942.1           | 6750.8      | 2.8            | -                | -           | -              |
| 8.67                 | 90         | 4000.0           | 4000.0      | 0.0            | 4000.0           | 4000.0      | 0.0            |
|                      | 100        | 4739.0           | 4739.1      | 0.0            | 4964.5           | 4964.5      | 0.0            |
|                      | 110        | 5939.6           | 5939.6      | 0.0            | 6057.7           | 6057.8      | 0.0            |
|                      | 116        | -                | -           | -              | 6603.4           | 6603.4      | 0.0            |
|                      | 119        | 6715.5           | 6716.0      | 0.0            | -                | -           | -              |



**Table 3.7. Comparison of Required Concrete Strength at Service**  
(Skew = 0°, Strand Diameter = 0.5 in.)

| Girder Spacing (ft.) | Span (ft.) | Standard         |             |                | LRFD             |             |                |
|----------------------|------------|------------------|-------------|----------------|------------------|-------------|----------------|
|                      |            | Updated <i>n</i> | TxDOT Meth. | Difference (%) | Updated <i>n</i> | TxDOT Meth. | Difference (%) |
| 6                    | 90         | 5000.0           | 5000.0      | 0.0            | 5000.0           | 5000.0      | 0.0            |
|                      | 100        | 5000.0           | 5000.0      | 0.0            | 5000.0           | 5000.0      | 0.0            |
|                      | 110        | 5000.0           | 5000.0      | 0.0            | 5000.0           | 5000.0      | 0.0            |
|                      | 120        | 5996.3           | 5955.5      | 0.7            | 5930.6           | 5930.6      | 0.0            |
|                      | 130        | 7498.4           | 7384.6      | 1.5            | 7578.9           | 6833.0      | 10.9           |
|                      | 133        | -                | -           | -              | 8554.7           | 8619.5      | -0.8           |
|                      | 136        | 8998.3           | 8621.6      | 4.4            | -                | -           | -              |
| 8                    | 90         | 5000.0           | 5000.0      | 0.0            | 5000.0           | 5000.0      | 0.0            |
|                      | 100        | 5000.0           | 5000.0      | 0.0            | 5000.0           | 5000.0      | 0.0            |
|                      | 110        | 5698.6           | 5583.9      | 2.1            | 5893.2           | 5893.2      | 0.0            |
|                      | 120        | 7293.0           | 7164.7      | 1.8            | 8394.8           | 7598.9      | 10.5           |
|                      | 124        | 8749.9           | 8306.4      | 5.3            | -                | -           | -              |
| 8.67                 | 90         | 5000.0           | 5000.0      | 0.0            | 5000.0           | 5000.0      | 0.0            |
|                      | 100        | 5000.0           | 5000.0      | 0.0            | 5000.0           | 5000.0      | 0.0            |
|                      | 110        | 5939.6           | 5939.6      | 0.0            | 6057.7           | 6057.8      | 0.0            |
|                      | 116        | -                | -           | -              | 7304.1           | 6780.5      | 7.7            |
|                      | 119        | 8155.0           | 7602.4      | 7.3            | -                | -           | -              |

### 3.2.7 Impact on Flexural Moment Resistance

The impact of not updating the modular ratio on the flexural moment resistance of the section is evaluated in this section. The results are presented in Table 3.8. The flexural moment resistance is found to be increasing in few cases when the modular ratio is updated. However, the increase is negligible. The increase in the flexural moment resistance is due to the increase in the number of strands and concrete strength at service.

**Table 3.8. Comparison of Flexural Moment Resistance,  $M_r$**   
**(Skew = 0°, Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Standard    |             |                | LRFD        |             |                |
|----------------------|------------|-------------|-------------|----------------|-------------|-------------|----------------|
|                      |            | Updated $n$ | TxDOT Meth. | Difference (%) | Updated $n$ | TxDOT Meth. | Difference (%) |
| 6                    | 90         | 4616.7      | 4616.7      | 0.0            | 4946.8      | 4946.8      | 0.0            |
|                      | 100        | 5962.4      | 5962.4      | 0.0            | 6273.2      | 6273.2      | 0.0            |
|                      | 110        | 6923.2      | 6923.2      | 0.0            | 7421.7      | 7421.7      | 0.0            |
|                      | 120        | 8400.9      | 8400.9      | 0.0            | 8870.0      | 8870.0      | 0.0            |
|                      | 130        | 10180.9     | 9959.0      | 2.2            | 10024.1     | 10004.7     | 0.2            |
|                      | 133        | -           | -           | -              | 10492.5     | 10391.0     | 1.0            |
|                      | 136        | 11256.0     | 10964.0     | 2.7            | -           | -           | -              |
| 8                    | 90         | 5728.7      | 5728.7      | 0.0            | 6059.9      | 6059.9      | 0.0            |
|                      | 100        | 7398.1      | 7398.1      | 0.0            | 7695.8      | 7695.8      | 0.0            |
|                      | 110        | 9228.8      | 8936.6      | 3.3            | 9489.1      | 9489.1      | 0.0            |
|                      | 120        | 11070.7     | 10836.0     | 2.2            | 11038.1     | 11018.7     | 0.2            |
|                      | 124        | 12139.1     | 11857.1     | 2.4            | -           | -           | -              |
| 8.67                 | 90         | 6099.1      | 6099.1      | 0.0            | 6430.4      | 6430.4      | 0.0            |
|                      | 100        | 7760.2      | 7760.2      | 0.0            | 8058.9      | 8058.9      | 0.0            |
|                      | 110        | 9589.7      | 9589.7      | 0.0            | 10113.7     | 10113.7     | 0.0            |
|                      | 116        | -           | -           | -              | 11081.4     | 11078.3     | 0.0            |
|                      | 119        | 11608.2     | 11608.2     | 0.0            | -           | -           | -              |

### 3.2.8 Impact on Shear Design

#### 3.2.8.1 Transverse Shear Design

The impact of not updating the modular ratio on the transverse shear reinforcement area is evaluated in this section. The results are presented in Table 3.9. The transverse shear reinforcement area is found to be decreasing in few cases when the modular ratio is updated. The decrease in the area of transverse reinforcement is due to the increase in the concrete strength at service, which consequently increases the shear capacity of concrete requiring lesser steel reinforcement.

**Table 3.9. Comparison of Transverse Shear Reinforcement Area,  $A_v$   
(Skew = 0°, Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Standard    |             |                | LRFD        |             |                |
|----------------------|------------|-------------|-------------|----------------|-------------|-------------|----------------|
|                      |            | Updated $n$ | TxDOT Meth. | Difference (%) | Updated $n$ | TxDOT Meth. | Difference (%) |
| 6                    | 90         | 0.12        | 0.13        | -8.7           | 0.13        | 0.13        | -0.1           |
|                      | 100        | 0.10        | 0.11        | -11.8          | 0.17        | 0.17        | -0.1           |
|                      | 110        | 0.08        | 0.09        | -15.0          | 0.20        | 0.20        | -0.1           |
|                      | 120        | 0.08        | 0.08        | 0.0            | 0.21        | 0.21        | -0.1           |
|                      | 130        | 0.08        | 0.08        | 0.0            | 0.18        | 0.21        | -16.0          |
|                      | 133        | -           | -           | -              | 0.19        | 0.17        | 12.3           |
|                      | 136        | 0.08        | 0.08        | 0.0            | -           | -           | -              |
| 8                    | 90         | 0.29        | 0.30        | -4.1           | 0.21        | 0.22        | -0.1           |
|                      | 100        | 0.27        | 0.29        | -4.5           | 0.26        | 0.26        | -0.1           |
|                      | 110        | 0.21        | 0.22        | -6.0           | 0.26        | 0.26        | -0.1           |
|                      | 120        | 0.08        | 0.08        | 0.0            | 0.24        | 0.27        | -12.0          |
|                      | 124        | 0.08        | 0.08        | 0.0            | -           | -           | -              |
| 8.67                 | 90         | 0.35        | 0.36        | -3.8           | 0.24        | 0.24        | 0.0            |
|                      | 100        | 0.34        | 0.34        | -0.5           | 0.28        | 0.28        | -0.1           |
|                      | 110        | 0.26        | 0.26        | 0.0            | 0.28        | 0.28        | -0.1           |
|                      | 116        | -           | -           | -              | 0.28        | 0.30        | -7.1           |
|                      | 119        | 0.08        | 0.08        | 0.0            | -           | -           | -              |

### 3.2.8.2 Interface Shear Design

The impact of not updating the modular ratio on the interface shear reinforcement area is evaluated in this section. The results are presented in Table 3.10. The interface shear reinforcement area remains the same for the Standard designs, and there is a very negligible effect on the LRFD designs.

**Table 3.10. Comparison of Transverse Interface Reinforcement Area,  $A_{vh}$**   
**(Skew = 0°, Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Standard    |             |                | LRFD        |             |                |
|----------------------|------------|-------------|-------------|----------------|-------------|-------------|----------------|
|                      |            | Updated $n$ | TxDOT Meth. | Difference (%) | Updated $n$ | TxDOT Meth. | Difference (%) |
| 6                    | 90         | 0.20        | 0.20        | 0.0            | 0.35        | 0.35        | 0.0            |
|                      | 100        | 0.20        | 0.20        | 0.0            | 0.42        | 0.42        | 0.0            |
|                      | 110        | 0.20        | 0.20        | 0.0            | 0.49        | 0.49        | 0.0            |
|                      | 120        | 0.20        | 0.20        | 0.0            | 0.60        | 0.60        | 0.0            |
|                      | 130        | 0.20        | 0.20        | 0.0            | 0.69        | 0.68        | 0.6            |
|                      | 133        | -           | -           | -              | 0.77        | 0.75        | 3.7            |
|                      | 136        | 0.20        | 0.20        | 0.0            | -           | -           | -              |
| 8                    | 90         | 0.20        | 0.20        | 0.0            | 0.55        | 0.55        | 0.0            |
|                      | 100        | 0.20        | 0.20        | 0.0            | 0.64        | 0.64        | 0.0            |
|                      | 110        | 0.20        | 0.20        | 0.0            | 0.73        | 0.73        | 0.0            |
|                      | 120        | 0.20        | 0.20        | 0.0            | 0.91        | 0.91        | -0.2           |
|                      | 124        | 0.20        | 0.20        | 0.0            | -           | -           | -              |
| 8.67                 | 90         | 0.20        | 0.20        | 0.0            | 0.61        | 0.61        | 0.0            |
|                      | 100        | 0.20        | 0.20        | 0.0            | 0.70        | 0.70        | 0.0            |
|                      | 110        | 0.20        | 0.20        | 0.0            | 0.80        | 0.80        | 0.0            |
|                      | 116        | -           | -           | -              | 0.92        | 0.92        | -0.5           |
|                      | 119        | 0.20        | 0.20        | 0.0            | -           | -           | -              |

### 3.2.9 Impact on Camber

The impact of not updating the modular ratio on camber is evaluated in this section. The results are presented in Table 3.11. The camber is found to be decreasing in few cases when the modular ratio is updated. The decrease in the camber is negligible and is caused due to the increase in the concrete strength at release, which consequently increases the elastic modulus of girder concrete, resulting in reduced camber.

**Table 3.11. Comparison of Camber**  
**(Skew = 0°, Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Standard         |             |                | LRFD             |             |                |
|----------------------|------------|------------------|-------------|----------------|------------------|-------------|----------------|
|                      |            | Updated <i>n</i> | TxDOT Meth. | Difference (%) | Updated <i>n</i> | TxDOT Meth. | Difference (%) |
| 6                    | 90         | 0.116            | 0.116       | 0.0            | 0.120            | 0.120       | 0.0            |
|                      | 100        | 0.175            | 0.175       | 0.0            | 0.195            | 0.195       | 0.0            |
|                      | 110        | 0.211            | 0.211       | 0.0            | 0.254            | 0.254       | 0.0            |
|                      | 120        | 0.283            | 0.283       | -0.1           | 0.343            | 0.343       | 0.0            |
|                      | 130        | 0.357            | 0.357       | 0.1            | 0.361            | 0.365       | -1.0           |
|                      | 133        | -                | -           | -              | 0.320            | 0.334       | -4.1           |
|                      | 136        | 0.329            | 0.339       | -3.0           | -                | -           | -              |
| 8                    | 90         | 0.161            | 0.161       | 0.0            | 0.177            | 0.177       | 0.0            |
|                      | 100        | 0.244            | 0.244       | 0.0            | 0.258            | 0.258       | 0.0            |
|                      | 110        | 0.339            | 0.321       | 5.5            | 0.348            | 0.348       | 0.0            |
|                      | 120        | 0.394            | 0.395       | -0.3           | 0.374            | 0.379       | -1.2           |
|                      | 124        | 0.361            | 0.374       | -3.6           | -                | -           | -              |
| 8.67                 | 90         | 0.179            | 0.179       | 0.0            | 0.195            | 0.195       | 0.0            |
|                      | 100        | 0.263            | 0.263       | 0.0            | 0.276            | 0.276       | 0.0            |
|                      | 110        | 0.355            | 0.355       | 0.0            | 0.362            | 0.362       | 0.0            |
|                      | 116        | -                | -           | -              | 0.384            | 0.387       | -0.9           |
|                      | 119        | 0.387            | 0.390       | -0.9           | -                | -           | -              |

## **4. PARAMETRIC STUDY OUTLINE**

### **4.1 GENERAL**

A parametric study was conducted for Type C and AASHTO Type IV single span, interior prestressed concrete bridge girders. Designs based on the AASHTO Standard Specifications (2002) were compared to designs based on the AASHTO LRFD Specifications (2004) for similar design parameters. The main focus of this parametric study was to evaluate the impact of the AASHTO LRFD Specifications on various design results including maximum span length, required number of strands, required concrete strengths at release and at service, flexural strength limit state, and shear design.

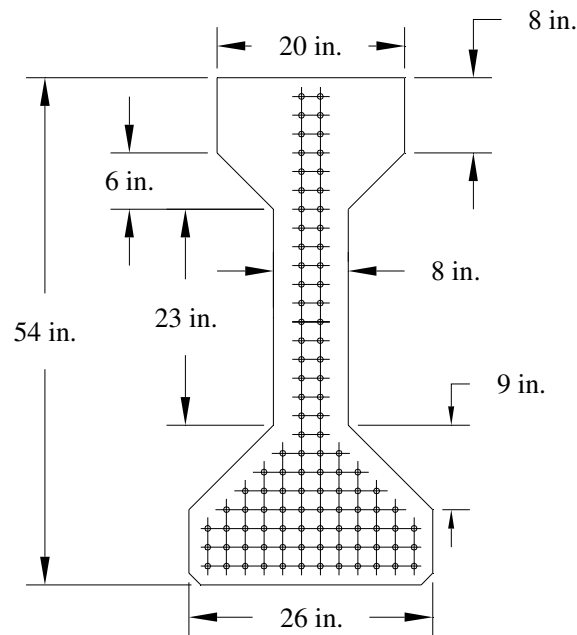
A design program was developed using Matlab 6.5.1 (Mathworks 2003) to carry out this task. The program can handle the design of both Type C and AASHTO Type IV girders according to the AASHTO Standard and the AASHTO LRFD Specifications. The results from the program were validated using TxDOT's PSTRS14 bridge design software (TxDOT 2004). A number of cases for a range of design parameters were evaluated.

The following sections describe the girder sections and their properties and discuss the methodology used in the design program developed for this study. The design of prestressed concrete girders essentially includes the service load design, ultimate flexural strength design, and shear design. The difference in each of the design procedures specified by the AASHTO Standard and LRFD Specifications are outlined. The assumptions made in the analysis and design are also discussed. The results from this parametric study for AASHTO Type IV girders are provided in Section 5 and for Type C girders in Section 6.

## 4.2 GIRDER SECTIONS

### 4.2.1 AASHTO Type IV Prestressed Concrete Bridge Girder

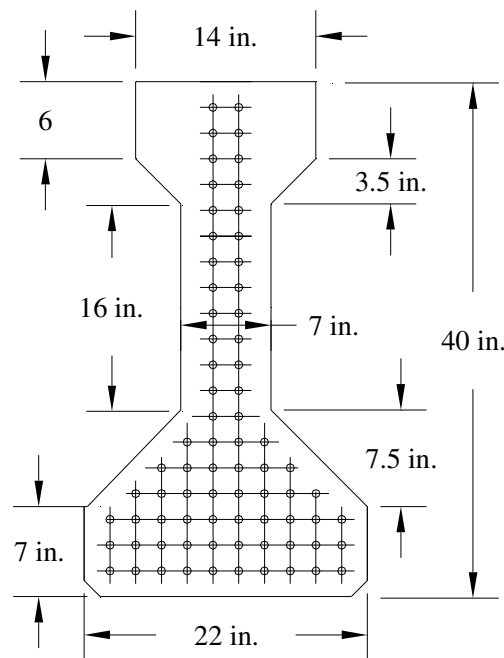
The AASHTO Type IV girder was introduced in 1968. Since then it has been one of the most economical shapes for prestressed concrete bridges. This girder type is used widely in Texas and in other states. The AASHTO Type IV girder can be used for bridges spanning up to 130 ft. with normal concrete strengths and is considered to be tough and stable. The girder is 54 in. deep having an I shaped cross-section. The top flange is 20 in. wide and the web thickness is 8 in. The fillets are provided between the web and the flanges to ensure a uniform transition of the cross section. The girder can hold a maximum of 102 strands. Both straight and harped strand patterns are allowed for this girder type. Figure 4.1 shows the details of AASHTO Type IV girder cross section.



**Figure 4.1. Configuration and Dimensions of the AASHTO Type IV Girder Section**  
[Adapted from TxDOT Bridge Design Manual (TxDOT 2001)].

#### 4.2.2 Type C Prestressed Concrete Bridge Girder

Type C girders are typically used in Texas for bridges spanning in the range of 40 to 90 ft. with normal concrete strengths. This is one of the earliest I shaped cross-section girders, first developed in 1957. It has been modified slightly since then in order to handle longer spans. The total depth of the girder is 40 in. with a 14 in. top flange and 7 in. thick web. The top flange is 6 in. thick and the bottom flange is 7 in. thick. The fillets are provided between the web and the flanges to ensure uniform transition of the cross section. The larger bottom flange allows an increased number of strands. The girder can hold a maximum of 74 strands. Both straight and harped strand patterns are allowed for this girder. Figure 4.2 shows the dimensions and configuration of the Type C girder cross-section.



**Figure 4.2. Configuration and Dimensions of the Type C Girder Section**  
[Adapted from TxDOT Bridge Design Manual (TxDOT 2001)].



### 4.3 DESIGN PROGRAM OUTLINE

A design program was developed using Matlab 6.5.1 (Mathworks 2003) to conduct the parametric study. The program is capable of handling Type C and AASHTO Type IV girder designs. The design program is consistent with the respective AASHTO Specifications with some modifications based on TxDOT design practice. The areas where modifications are made are discussed in the following sections. The design program consists of a driver program “mainprog.m” which calls the other functions. The first function called is “readingdata.m”. This function reads the input data from excel sheet “input1.xls” or from Matlab command line. Sample input for the program is shown in Table 4.1.

**Table 4.1. Sample Input for Design Program “mainprog.m”.**

|  |      |
|--|------|
| Girder Type (1 for Type C and 2 for Type IV)                   | 2    |
| Specifications (1 for Standard and 2 for LRFD)                 | 2    |
| Span Length, ft. (c/c pier)                                    | 90   |
| Girder Spacing, ft.  | 8.67 |
| Strand Diameter, in.   | 0.5  |
| Concrete Strength at release, psi (0 to optimize)              | 0    |
| Concrete Strength at service, psi (0 to optimize)              | 0    |
| Prestress losses (1 for TxDOT and 2 for LRFD methodology)      | 2    |
| Relative Humidity, %   | 60   |
| Skew angle (degrees)   | 0    |
| Output form (1 for output in excel, 2 for command line output) | 1    |

The modular ratio is evaluated based on input concrete strengths. The modular ratio is assumed to be 1 if the input for concrete strengths is 0, and the final concrete strengths at release and at service are optimized using TxDOT’s methodology (TxDOT 2001). The program does not consider the haunch effect based on TxDOT’s recommendations (TxDOT 2001). The number of girders in the bridge cross section is established based on a total bridge width of 46'-0" and a clear roadway width of 44'-0".

The program assigns the design variables based on the Specifications under consideration. The design variables considered in the design are presented in Table 4.2.

**Table 4.2. Design Variables for AASHTO Standard and LRFD Designs.**

| Category             | Specifications    | Description                                | Proposed Value  |
|----------------------|-------------------|--|---|
| Prestressing Strands | Standard          | Ultimate Strength, $f'_s$                  | 270 ksi – low-relaxation  |
|                      |                   | Jacking Stress Limit, $f_{si}$             | $0.75 f'_s$   |
|                      |                   | Yield Strength, $f_y$                      | $0.9 f'_s$  |
|                      |                   | Modulus of Elasticity, $E_s$               | 28000 ksi   |
|                      | LRFD              | Ultimate Strength, $f_{pu}$                | 270 ksi – low-relaxation  |
|                      |                   | Jacking Stress Limit, $f_{pi}$             | $0.75 f_{pu}$   |
|                      |                   | Yield Strength, $f_{py}$                   | $0.9 f_{pu}$  |
|                      |                   | Modulus of Elasticity, $E_p$               | 28500 ksi   |
| Concrete-Precast     | Standard and LRFD | Unit Weight, $w_c$                         | 150 pcf   |
|                      |                   | Modulus of Elasticity, $E_c$               | $33 w_c^{1.5} \sqrt{f'_c}$ ( $f'_c$ precast)  |
| Concrete-CIP Slab    | Standard and LRFD | Slab Thickness, $t_s$                      | 8 in.   |
|                      |                   | Unit Weight, $w_c$                         | 150 pcf   |
|                      |                   | Modulus of Elasticity, $E_{cip}$           | $33 w_c^{1.5} \sqrt{f'_c}$ ( $f'_c$ CIP)  |
|                      |                   | Specified Compressive Strength ( $f'_c$ )  | 4000 psi  |
|                      |                   | Modular Ratio, $n$                         | $E_{cip}/E_c$   |
| Other                | Standard and LRFD | Relative Humidity                          | 60%   |
|                      |                   | Non-Composite Dead Loads                   | 1.5" asphalt wearing surface<br>(Unit weight of 140 pcf)  |
|                      |                   | Composite Dead Loads                       | T501 type rails (326 plf)   |
|                      |                   | Harping in AASHTO Type IV & Type C Girders | An allowable harping pattern consistent with TxDOT practices will be selected to limit the initial stresses to the required values. |

The main driver program “mainprog.m” calls one of the following functions based on the input data.

1. typeCstd.m: This function handles the design for Type C girders based on AASHTO Standard Specifications
2. typeClrfd.m: This function handles the design for Type C girders based on AASHTO LRFD Specifications
3. type4std.m: This function handles the design for Type IV girders based on AASHTO Standard Specifications
4. type4lrfd.m: This function handles the design for Type IV girders based on AASHTO LRFD Specifications

## **4.4 DESIGN ASSUMPTIONS AND PROCEDURE**

### **4.4.1 General**

The analysis and design procedure followed for girder designs based on the AASHTO Standard Specifications (2002) and the AASHTO LRFD Specifications (2004) are discussed in this section. Modifications made by TxDOT in the design are also included.

### **4.4.2 Member Properties**

The non-composite and transformed composite section properties of the girders are evaluated as discussed in the following sections.

#### *4.4.2.1 Non-Composite Section Properties*

The non-composite section properties for each type of girder as specified by the TxDOT Design Manual (TxDOT 2001) are presented in Table 4.3. These properties are the same irrespective of the specifications used.

**Table 4.3. Non-Composite Section Properties.**

|         | $y_t$ (in.) | $y_b$ (in.) | Area (in. <sup>2</sup> ) | $I$ (in. <sup>4</sup> ) |
|---------|-------------|-------------|--------------------------|-------------------------|
| Type IV | 29.25       | 24.75       | 788.4                    | 260,403.0               |
| Type C  | 22.91       | 17.09       | 494.9                    | 82,602.0                |

where:

$I$  = Moment of inertia about the centroid of the non-composite precast girder, in.<sup>4</sup>

$y_b$  = Distance from centroid to the extreme bottom fiber of the non-composite precast girder, in.

$y_t$  = Distance from centroid to the extreme top fiber of the non-composite precast girder, in.

#### 4.4.2.2 Composite Section Properties

The composite section properties depend on the effective flange width of the girder. AASHTO Standard Specifications Article 9.8.3.2 specifies the effective flange width of an interior girder to be the least of the following:

1. One-fourth of the span length of the girder,
2.  $6 \times$  (slab thickness on each side of the effective web width) + effective web width, or
3. One-half the clear distance on each side of the effective web width plus the effective web width.

The effective web width used in conditions (2) and (3) is specified by AASHTO Standard Article 9.8.3.1, as the lesser of the following:

1.  $6 \times$  (flange thickness on either side of web) + web thickness + fillets, and
2. Width of the top flange.

The AASHTO LRFD Specifications specify a slightly modified approach as compared to Standard Specifications for the calculation of effective flange width of interior girders. The LRFD Specifications does not require the calculation of the effective web width and instead uses the greater of actual web thickness and one-half of

the girder top flange width in condition (2) given below. LRFD Article 4.6.2.6.1 specifies the effective flange width for an interior girder to be the least of the following:

1. One-fourth of the effective span length,
2.  $12 \times (\text{average slab thickness}) + \text{greater of web thickness or one-half the girder top flange width}$ , or
3. The average spacing of adjacent girders.

Once the effective flange width is established, the transformed flange width and flange area is calculated as

$$\text{Transformed flange width} = n \times (\text{effective flange width}) \quad (4.1)$$

$$\text{Transformed flange Area} = n \times (\text{effective flange width}) (t_s) \quad (4.2)$$

where:

$$n = \text{Modular ratio between slab and girder concrete} = E_{cip}/E_c$$

$$t_s = \text{Thickness of the slab, in.}$$

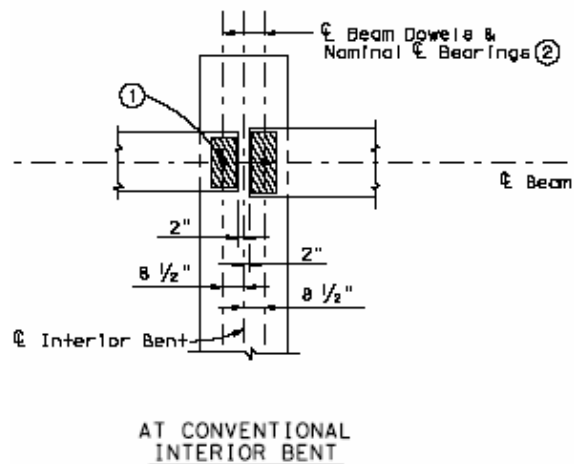
$$E_{cip} = \text{Modulus of elasticity of cast in place slab concrete, ksi}$$

$$E_c = \text{Modulus of elasticity of precast girder concrete, ksi}$$

TxDOT recommends using the modular ratio as 1 because the concrete strengths are unknown at the beginning of the design process and are optimized during the design. This recommendation was followed for the service load design in this study. For shear and deflection calculations the actual modular ratio based on the selected optimized precast concrete strength is used in this study. For these calculations the composite section properties are evaluated using the transformed flange width and precast section properties. The flexural strength calculations are based on the selected optimized precast concrete strength, the actual slab concrete strength, and the actual slab and girder dimensions.

#### 4.4.2.3 Design Span Length, Hold-Down Point and Critical Section for Shear

The design span length is the center-to-center distance between the bearings. This length is obtained by deducting the distance between the centerlines of the bearing pad and the pier from the total span length (center-to-center distance between the piers). Figure 4.3 illustrates the details at the girder end at a conventional support. The hold-down point for the harped strands is specified by the TxDOT Bridge Design Manual (TxDOT 2001) to be the greater of 5 ft. and 0.05 times the span length, on either side of the midspan.



**Figure 4.3. Girder End Details (TxDOT Standard Drawings 2001).**

The critical section for shear is specified by AASHTO Standard Specifications as the distance  $h/2$  from the face of the support, where  $h$  is the depth of the composite section. However, as the support dimensions are not specified in this study, the critical section is measured from the centerline of bearing, which yields a conservative estimate of the design shear force. The LRFD Specifications requires the critical section for shear to be calculated based on the parameter  $\theta$  evaluated in the shear design section. The initial estimate for the location of the critical section for shear is taken as the distance

equal to  $h/2$  plus one-half the bearing pad width, from the girder end, where  $h$  is the depth of the composite section. The critical section is then refined based on an iterative process that determines the final values of the parameters  $\theta$  and  $\beta$ .

#### **4.4.3 Design Loads, Bending Moments and Shear Forces**

##### *4.4.3.1 General*

The dead and superimposed dead loads considered in the design are girder self weight, slab weight, barrier and asphalt wearing surface loads. The load due to the barrier and asphalt wearing surface are accounted for as composite loads (loads occurring after the onset of composite action between the deck slab and the precast girder section). The girder self weight and the slab weight are considered as non-composite loads. The live loads are consistent with the specifications under consideration. The impact and distribution factors are calculated as specified by the respective design specifications. The loads due to extreme events such as earthquake and vehicle collision are not considered in the design as they are not a design factor for bridges in Texas. The wind load is not taken into account for this study. The loads and load combinations specified by the AASHTO Standard and LRFD Specifications are discussed in the following sections.

##### *4.4.3.2 Dead Loads*

The dead load on the non-composite section is taken as the self weight of the girder. The self weight is taken as 0.821 klf for AASHTO Type IV girders and 0.516 klf for Type C girders as specified by the TxDOT Bridge Design Manual (TxDOT 2001).

##### *4.4.3.3 Superimposed Dead Loads*

The superimposed dead load on the non-composite section is due to the slab weight. The unit weight of slab concrete is taken as 150 pcf. The tributary width for calculating the slab load is taken as the center-to-center spacing between the adjacent girders. The superimposed dead loads on the composite section are the weight of the

barrier, and the asphalt wearing surface weight. TxDOT recommended using the unit weight of the asphalt wearing surface as 140 pcf, and the barrier weight as 326 plf.

The Standard Specifications allows the superimposed dead loads on the composite section to be distributed equally among all the girders for all the design cases. The LRFD Specifications allows the equal distribution of the composite superimposed dead loads (permanent loads) only when the following conditions specified by LRFD Article 4.6.2.2.1. are satisfied:

1. Width of deck is constant,
2. Number of girders ( $N_b$ ) is not less than four,
3. Girders are parallel and have approximately the same stiffness,
4. The roadway part of the overhang,  $d_e \leq 3.0$  ft.,
5. Curvature in plan is less than 3 degrees for 3 or 4 girders and less than 4 degrees for 5 or more girders, and
6. Cross section of the bridge is consistent with one of the cross sections given in LRFD Table 4.6.2.2.1-1.

If the above conditions are not satisfied, refined analysis is required to determine the actual load on each girder. Grillage analysis and finite element analysis are recommended by the LRFD Specifications as appropriate refined analysis methods.

#### *4.4.3.4 Shear Force and Bending Moment due to Dead and Superimposed Dead Loads*

The bending moment ( $M$ ) and shear force ( $V$ ) due to dead loads and superimposed dead loads at any section having a distance  $x$  from the support, are calculated using the following formulas.

$$M = 0.5wx (L - x) \quad (4.3)$$

$$V = w(0.5L - x) \quad (4.4)$$

where:

$w$  = Uniform load, k/ft.

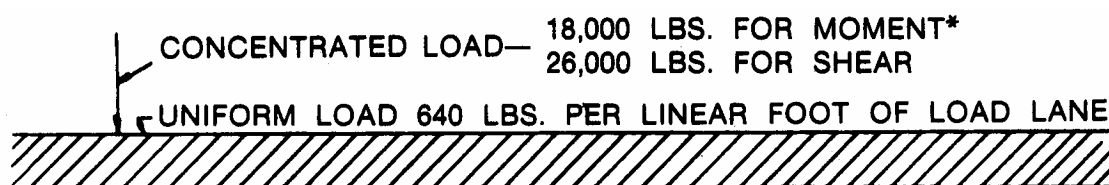
$L$  = Design span length, ft.



#### 4.4.3.5 Live Load

There is a significant change in the live load specified by the LRFD Specifications as compared to the Standard Specifications. The Standard Specifications specify the live load to be taken as one of the following, whichever produces maximum stresses at the section considered.

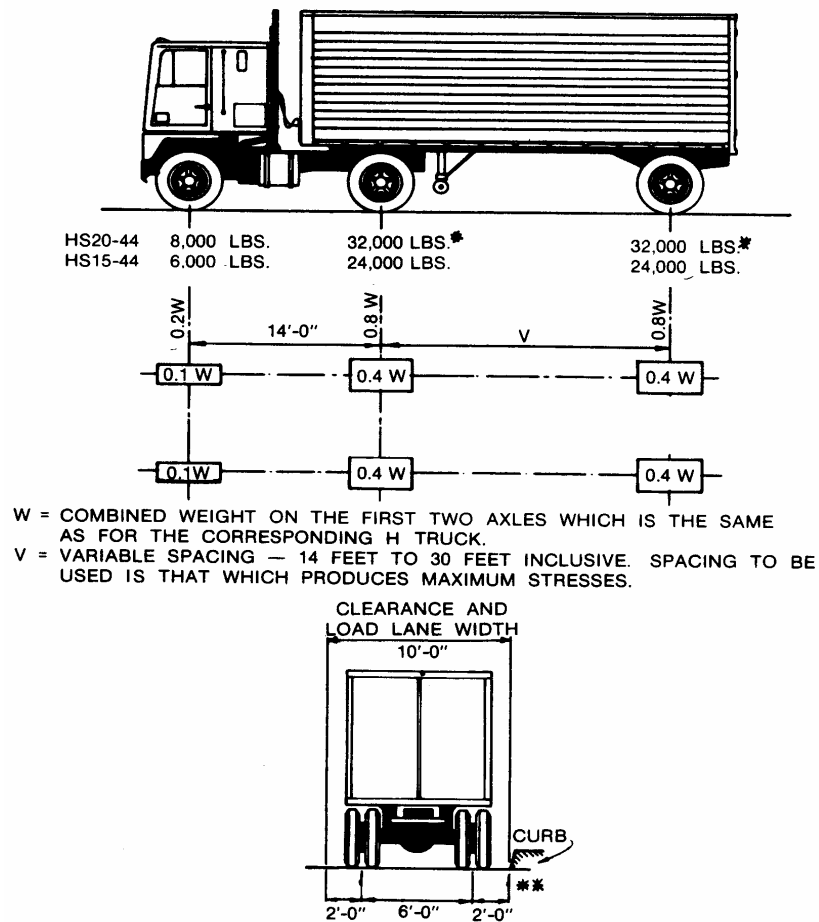
1. HS 20-44 truck consisting of one front axle weighing 8 kips and two rear axles weighing 32 kips each. The truck details are shown in Figure 4.5.
2. HS 20-44 lane loading consisting of 0.64 klf distributed load and a point load traversing the span having a magnitude of 18 kips for moment and 26 kips for shear. The details are shown in Figure 4.4.
3. Tandem loading consisting of two 24 kips axles spaced 4 ft. apart.



**Figure 4.4. HS 20-44 Lane Loading (AASHTO Standard Specifications 2002).**

The LRFD Specifications specify a new live load model. The live load is to be taken as one of the following, whichever yields maximum stresses at the section considered.

1. HL-93: This is a combination of an HS 20-44 truck consisting of one front axle weighing 8 kips and two rear axles weighing 32 kips each with a 0.64 klf uniformly distributed lane load.
2. Combination of a tandem loading consisting of two 25 kips axles spaced 4 ft. apart with a 0.64 klf distributed lane load.



**Figure 4.5. HS 20-44 Truck Configuration  
(AASHTO Standard Specifications 2002)**

#### 4.4.3.6 Undistributed Live Load Shear and Moment

The undistributed shear force ( $V$ ) and bending moment ( $M$ ) due to HS 20-44 truck load, HS 20-44 lane load, and tandem load on a per-lane-basis are calculated using the following equations prescribed by the PCI Design Manual (PCI 2003).

Maximum bending moment due to HS 20-44 truck load.

For  $x/L = 0 - 0.333$

$$M = \frac{72(x)[(L - x) - 9.33]}{L} \quad (4.5)$$

For  $x/L = 0.333 - 0.50$

$$M = \frac{72(x)[(L - x) - 4.67]}{L} - 112 \quad (4.6)$$

Maximum shear force due to HS 20-44 truck load.

For  $x/L = 0 - 0.50$

$$V = \frac{72[(L - x) - 9.33]}{L} \quad (4.7)$$

Maximum bending moment due to HS 20-44 lane loading.

$$M = \frac{P(x)(L - x)}{L} + 0.5(w)(x)(L - x) \quad (4.8)$$

Maximum shear force due to HS 20-44 lane load.

$$V = \frac{Q(L - x)}{L} + (w)\left(\frac{L}{2} - x\right) \quad (4.9)$$

Maximum bending moment due to AASHTO LRFD lane load.

$$M = 0.5(w)(x)(L - x) \quad (4.10)$$

Maximum shear force due to AASHTO LRFD lane load.

$$V = \frac{0.32(L - x)^2}{L} \text{ for } x \leq 0.5L \quad (4.11)$$

Maximum bending moment due to Tandem load.

$$M = \frac{T(x)[(L - x) - 2]}{L} \quad (4.12)$$

Maximum shear force due to Tandem load.

$$V = \frac{T[(L - x) - 2]}{L} \quad (4.13)$$

where:

$M$  = Live load moment, k-ft.

$V$  = Live load shear, kips

$x$  = Distance from the support to the section at which bending moment or shear force is calculated, ft.

- $L$  = Design span length, ft.  
 $P$  = Concentrated load for moment = 18 kips  
 $Q$  = Concentrated load for shear = 26 kips  
 $w$  = Uniform load per linear foot of load lane = 0.64 klf  
 $T$  = Tandem load, 48 kips for AASHTO Standard and 50 kips for AASHTO LRFD design.

#### 4.4.3.7 Fatigue Load

The fatigue load for calculating the fatigue stress is given by LRFD Article 3.6.1.4 as a single HS 20-44 truck load with constant spacing of 30.0 ft. between the 32.0 kip rear axles.

#### 4.4.3.8 Undistributed Fatigue Load Moment

The undistributed bending moment ( $M$ ) due to fatigue load on a per-lane-basis is calculated using the following equations prescribed by the PCI Design Manual (PCI 2003)

Maximum bending moment due to fatigue truck load.

For  $x/L = 0 - 0.241$

$$M = \frac{72(x)[(L - x) - 18.22]}{L} \quad (4.14)$$

For  $x/L = 0.241 - 0.50$

$$M = \frac{72(x)[(L - x) - 11.78]}{L} - 112 \quad (4.15)$$

where:

- $x$  = Distance from the support to the section at which bending moment or shear force is calculated, ft.  
 $L$  = Design span length, ft.

#### 4.4.3.9 Impact and Distribution Factors

The AASHTO Standard and LRFD Specifications require the effect of dynamic (impact) loading to be considered. The dynamic load is expressed as a percentage of live load. AASHTO Standard Article 3.8.2.1 specifies the following expression to determine the impact load factor

$$I = \frac{50}{L + 125} \leq 30\% \quad (4.16)$$

where:

- $I$  = Impact factor
- $L$  = Design span length, ft.

AASHTO LRFD Article 3.6.2 specifies the dynamic load to be taken as 33 percent of the live load for all limit states except the fatigue limit state for which the impact factor is specified as 15 percent of the fatigue load moment. The impact factor for the Standard Specifications is applicable to truck, lane and tandem loads, however the LRFD Specifications do not require the lane loading to be increased for dynamic effects.

The live load moments and shear forces including the dynamic load (impact load) effect are distributed to the girders using the distribution factors. The Standard Specifications recommend using a live load moment distribution factor of  $S/11$  for prestressed concrete girders, where  $S$  is the girder spacing in ft. The distribution factor for live load shear varies with the position of the load. The TxDOT Bridge Design Manual (TxDOT 2001) recommends using  $S/11$  as the live load shear distribution factor. The Standard Specifications only consider the effect of girder spacing on the distribution factors and neglects the effect of other critical parameters such as slab stiffness, girder stiffness, and span length.

The LRFD Specifications provide more complex formulas for the distribution of live load moments and shear forces to individual girders. For skewed bridges, LRFD Specifications require the distribution factors for moment to be reduced and the shear distribution factors shall be corrected for skew. LRFD Table 4.6.2.2.2 and 4.6.2.2.3 specify the distribution factors for moment and shear, respectively. The use of these approximate distribution factors is allowed for prestressed concrete girders having an I-shaped cross section with composite slab, if the conditions outlined below are satisfied.

1. Width of deck is constant
2. Number of girders ( $N_b$ ) is not less than four (Lever rule can be used for 3 girders)
3. Girders are parallel and of approximately the same stiffness
4. The roadway part of the overhang,  $d_e \leq 3.0$  ft.
5. Curvature in plan is less than 3 degrees for 3 or 4 girders and less than 4 degrees for 5 or more girders
6. Cross-section of the bridge is consistent with one of the cross-sections given in LRFD Table 4.6.2.2.1-1.
7.  $3.5 \leq S \leq 16$  where  $S$  is the girder spacing, ft.
8.  $4.5 \leq t_s \leq 12$  where  $t_s$  is the slab thickness, in.
9.  $20 \leq L \leq 240$  where  $L$  is the span length, ft.
10.  $10,000 \leq K_g \leq 7,000,000$ , in.<sup>4</sup>

where:

$$K_g = n (I + A e_g^2)$$

$$n = \text{Modular ratio between the girder and slab concrete} = E_c / E_{cip}$$

$$E_{cip} = \text{Modulus of elasticity of cast in place slab concrete, ksi}$$

$$E_c = \text{Modulus of elasticity of precast girder concrete, ksi}$$

$$I = \text{Moment of inertia of the girder section, in.}^4$$

$$A = \text{Area of the girder cross section, in.}^2$$

$$e_g = \text{Distance between the centroids of the girder and the slab, in.}$$

For bridge configurations not satisfying the limits mentioned above, refined analysis is required to estimate the moment and shear distribution factors.

The distribution factors shall be taken as the greater of the two cases when two design lanes are loaded and one design lane is loaded. The approximate live load moment distribution factors (*DFM*) and the live load shear distribution factors (*DFV*) for an interior I-shaped girder cross-section with a composite slab (type k) is given by AASHTO LRFD Tables 4.6.2.2.2 and 4.6.2.2.3 as follows.

For two or more lanes loaded:

$$DFM = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12.0L t_s^3}\right)^{0.1} \quad (4.17)$$

For one design lane loaded:

$$DFM = 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12.0L t_s^3}\right)^{0.1} \quad (4.18)$$

For two or more lanes loaded:

$$DFV = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^2 \quad (4.19)$$

For one design lane loaded:

$$DFV = 0.36 + \left(\frac{S}{25.0}\right) \quad (4.20)$$

where:

*DFM* = Distribution factor for moment

*DFV* = Distribution factor for shear

*S* = Girder spacing, ft.

*L* = Design span length, ft.

*t<sub>s</sub>* = Thickness of slab, in.

*K<sub>g</sub>* = Longitudinal stiffness parameter, in.<sup>4</sup>

$$= n (I + A e_g^2)$$

- $n$  = Modular ratio between the girder and slab concrete  
 $I$  = Moment of inertia of the girder section, in.<sup>3</sup>  
 $A$  = Area of the girder cross section, in.<sup>2</sup>  
 $e_g$  = Distance between the centroids of the girder and the slab, in.

The distribution factor for fatigue load moment is to be taken as

$$DFM_f = \frac{DFM \text{ (single lane loaded)}}{m} \quad (4.21)$$

where:

- $DFM_f$  = Distribution factor for fatigue load moment  
 $m$  = Multiple presence factor taken as 1.2.

The live load moment distribution factors shall be reduced for skew using the skew reduction formula specified by AASHTO LRFD Article 4.6.2.2.2e. The skew reduction formula is applicable to any number of design lanes loaded. The skew reduction formula for prestressed concrete I shaped (type k) girders can be used when the following conditions are satisfied.

1.  $30^\circ \leq \theta \leq 60^\circ$  where  $\theta$  is the skew angle, if  $\theta > 60^\circ$ , use  $\theta = 60^\circ$
2.  $3.5 \leq S \leq 16$  where  $S$  is the girder spacing, ft.
3.  $20 \leq L \leq 240$  where  $L$  is the span length, ft.
4. Number of girders ( $N_b$ ) is not less than four

The skew reduction ( $SR$ ) is given as

$$SR = 1 - c_I(\tan \theta)^{1.5} \quad (4.22)$$

where:

$$c_I = 0.25 \left( \frac{K_g}{12.0 L t_s^3} \right)^{0.25} \left( \frac{S}{L} \right)^{0.5} \quad (4.23)$$

if  $\theta < 30^\circ$ ,  $c_I = 0.0$



The approximate live load shear distribution factors for interior girders shall be corrected for skew using the skew correction factors specified by LRFD Table 4.6.2.2.3c-1. The skew reduction formula is applicable to any number of design lanes loaded. The skew correction formula for prestressed concrete I shaped (type k) girders can be used when the following conditions are satisfied.

1.  $0^\circ \leq \theta \leq 60^\circ$  where  $\theta$  is the skew angle
2.  $3.5 \leq S \leq 16$  where  $S$  is the girder spacing, ft.
3.  $20 \leq L \leq 240$  where  $L$  is the span length, ft.
4. Number of girders ( $N_b$ ) is not less than four

The skew correction ( $SC$ ) is given as

$$SC = SC = 1.0 + 0.20 \left( \frac{12.0 L t_s^3}{K_g} \right)^{0.3} \tan \theta \quad (4.24)$$

#### 4.4.3.10 Distributed Live Load Shear Force and Bending Moment

The governing live load for the designs based on the AASHTO Standard Specifications is determined based on undistributed live load moments. The shear force at the critical section and bending moment at the midspan of the girder due to the governing live load, including the impact load, is calculated using the following formulas.

$$M_{LL+I} = (M) (DF) (1+I) \quad (4.25)$$

$$V_{LL+I} = (V) (DF) (1+I) \quad (4.26)$$

where:

- $M_{LL+I}$  = Distributed governing live load moment including impact loading, k-ft.
- $V_{LL+I}$  = Distributed governing live load shear including impact loading, kips
- $M$  = Governing live load bending moment per lane, k-ft.
- $V$  = Governing live load shear force per lane, kips

$DF$  = Distribution factor specified by the Standard Specifications.

$I$  = Impact factor specified by the Standard Specifications.

For the designs based on LRFD Specifications, the shear force at the critical section and bending moment at midspan is calculated for the governing (HS 20-44 truck or tandem) load and lane load separately. The governing load is based on undistributed tandem and truck load moments. The effect of dynamic loading is included only for the truck or tandem loading and not for lane loading. The formulas used in the design are as follows

$$M_{LT} = (M_T)(DFM)(1+IM) \quad (4.27)$$

$$V_{LT} = (V_T)(DFV)(1+IM) \quad (4.28)$$

$$M_{LL} = (M_L)(DFM) \quad (4.29)$$

$$V_{LL} = (V_L)(DFV) \quad (4.30)$$

$$M_{LL+I} = M_{LT} + M_{LL} \quad (4.31)$$

$$V_{LL+I} = V_{LT} + V_{LL} \quad (4.32)$$

$$M_f = (M_{fatigue})(DFM_f)(1+IM_f) \quad (4.33)$$

where:

$M_{LL+I}$  = Distributed moment due to live load including dynamic load effect, k-ft.

$V_{LL+I}$  = Distributed shear due to live load including dynamic load effect, kips

$M_{LT}$  = Distributed moment due to governing (truck or tandem) load including dynamic load effect, k-ft.

$M_T$  = Bending moment per lane due to governing (truck or tandem) load, k-ft.

$V_{LT}$  = Distributed shear due to governing (truck or tandem) load including dynamic load effect, kips

$V_T$  = Shear force per lane due to governing (truck or tandem) load, kips

$M_{LL}$  = Distributed moment due to lane load, k-ft.

$M_L$  = Bending moment per lane due to lane load, k-ft.

|               |   |   |
|---------------|---|---|
| $V_{LL}$      | = | Distributed shear due to lane load, kips                                    |
| $V_L$         | = | Shear force per lane due to lane load, kips                                 |
| $M_f$         | = | Distributed moment due to fatigue load including dynamic load effect, k-ft. |
| $M_{fatigue}$ | = | Bending moment per lane due to fatigue load, k-ft.                          |
| $DFM$         | = | Moment distribution factor specified by LRFD Specifications                 |
| $DFV$         | = | Shear distribution factor specified by LRFD Specifications                  |
| $IM$          | = | Impact factor specified by LRFD Specifications                              |
| $DFM_f$       | = | Moment distribution factor for fatigue loading                              |
| $IM_f$        | = | Impact factor for fatigue limit state                                       |

#### 4.4.3.11 Load Combinations

Significantly different loads combinations are specified by the LRFD Specifications as compared to the Standard Specifications. The major difference occurred due to the different methodologies followed by the two codes. The Standard Specifications uses the Service Load Design (SLD) method for the load combinations at service limit state and Load Factor Design (LFD) for load combinations at ultimate strength limit state. The LRFD Specifications uses the Load Resistance Factor Design (LRFD) method for strength load combination. The Service I and Strength I load combinations specified by both the Standard and LRFD Specifications are applicable for prestressed concrete girders. The Service I load combination is applicable for all types of members including prestressed concrete girders. This load combination is used to check the compressive and tensile stresses due to service loads for designs based on Standard Specifications. For designs based on the LRFD Specifications the Service I load combination is used to check only the compressive stresses. The Strength I load combination is used to check the shear capacity and ultimate flexural capacity of the member.

The AASHTO LRFD Specifications specifies Service III and Fatigue load combinations for prestressed concrete members in addition to the Service I and Strength

I load combinations. Service III load combination is exclusively applicable to prestressed concrete members to check tensile stresses at the bottom fiber of the girder. The objective of this load combination is to prevent cracking of prestressed concrete members. The Fatigue load combination is used to check the fatigue of prestressing strands due to repetitive vehicular live load.

Extreme events, such as earthquake loads and vehicle collision loads are not accounted for in this parametric study. The wind load is also not considered as this does not governs the design of bridges in Texas. The applicable load combinations including dead, superimposed and live loads specified by AASHTO Standard Table 3.22.1A are outlined as follows

For service load design (Group I):

$$Q = 1.00D + 1.00(L+I) \quad (4.34)$$

For load factor design (Group I):

$$Q = 1.3[1.00D + 1.67(L+I)] \quad (4.35)$$

where:

$Q$  = Factored load effect

$D$  = Dead load effect

$L$  = Live load effect

$I$  = Impact load effect

The load combinations specified by AASHTO LRFD Table 3.4.1-1 are outlined as follows

Service I - checks compressive stresses in prestressed concrete components:

$$Q = 1.00(DC + DW) + 1.00(LL + IM) \quad (4.36)$$

where:

- $Q$  = Total load effect
- $DC$  = Self weight of girder and attachment (slab and barrier) load effect
- $DW$  = Wearing surface load effect
- $LL$  = Live load effect
- $IM$  = Dynamic load effect

Service III - checks tensile stresses in prestressed concrete components:

$$Q = 1.00(DC + DW) + 0.80(LL + IM) \quad (4.37)$$

Strength I - checks ultimate strength:

$$\text{Maximum } Q = 1.25(DC) + 1.50(DW) + 1.75(LL + IM) \quad (4.38)$$

$$\text{Minimum } Q = 0.90(DC) + 0.65(DW) + 1.75(LL + IM) \quad (4.39)$$

For simple span bridges, the maximum load factors produce maximum effects. However, minimum load factors are used for dead load ( $DC$ ) and wearing surface load ( $DW$ ) when dead load and wearing surface stresses are opposite to those of the live load. For the present study involving simply supported bridge girders, only the maximum load combination is applicable.

Fatigue - checks stress range in strands:

$$Q = 0.75(LL + IM) \quad (4.40)$$

#### **4.4.4 Service Load Design Calculations**

##### *4.4.4.1 General*

The flexural design of prestressed concrete bridge girders is generally controlled by the service limit state, while the strength limit state seldom controls the design. However, the strength limit state needs to be checked to ensure safety at ultimate load

conditions. The steps involved in the service load design and the procedures specified by the AASHTO Standard and LRFD Specifications are outlined in this section.

#### 4.4.4.2 Service Load Stresses

The tensile stress at the bottom fiber of the girder at midspan due to external loading is evaluated using the Service I limit state load combination for the Standard Specifications and the Service III limit state for the LRFD Specifications. This limit state often controls the service load design of prestressed concrete members and is used for the preliminary design. The formulas used are as follows, with the construction considered to be unshored.

For the Standard Specifications:

$$f_b = \frac{M_g + M_s}{S_b} + \frac{M_{SDL} + M_{LL+I}}{S_{bc}} \quad (4.41)$$

where:

- $f_b$  = Concrete stress at the bottom fiber of the girder due to applied loads, ksi
- $M_g$  = Unfactored bending moment due to girder self-weight, k-in.
- $M_s$  = Unfactored bending moment due to slab weight, k-in.
- $M_{SDL}$  = Unfactored bending moment due to superimposed dead loads (barrier and asphalt wearing surface), k-in.
- $M_{LL+I}$  = Distributed bending moment due to live load including impact, k-in.
- $S_b$  = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder, in.<sup>3</sup>
- $S_{bc}$  = Composite section modulus referenced to the extreme bottom fiber of the precast girder, in.<sup>3</sup>

For the LRFD Specifications:

$$f_b = \frac{M_g + M_s}{S_b} + \frac{M_{SDL} + (0.8)(M_{LT} + M_{LL})}{S_{bc}} \quad (4.42)$$

where:

$M_{LT}$  = Distributed bending moment due to governing (truck or tandem) load including impact load, k-in.

$M_{LL}$  = Distributed bending moment due to lane load, k-in.

The additional variables were defined for Equation 4.41.

The stress at the bottom fiber due to service loads is then compared with the allowable tensile stress at service load. Based on the difference between the two, a preliminary estimate of the required prestressing force is made.

#### 4.4.4.3 Preliminary Estimate of Required Prestress

The preliminary estimate of the required prestress is made once the maximum tensile stress due to service loads at the bottom fiber of the girder is calculated. The difference between the maximum tensile stress and the allowable tensile stress at the bottom fiber gives the required stress due to prestressing. Assuming the total prestress losses of 20 percent and the eccentricity of the strands at the midspan equal to the distance from the centroid of the girder to the bottom fiber, an estimate of required number of strands is made. For this number of strands the actual midspan eccentricity and bottom fiber stress due to prestressing is calculated. The number of strands is incremented by two in each trial until the final bottom fiber stress satisfies the allowable stress limits. The strands are placed as low as possible on the grid shown in Figures 4.1 and 4.2, each row filled before proceeding to the next higher row (TxDOT PSTRS14 Guide, TxDOT 2005). The bottom fiber stress at the midspan due to the prestressing force is calculated using the following formula.

$$f_{bp} = \frac{P_{se}}{A} + \frac{P_{se} e_c}{S_b} \quad (4.43)$$

where:

- $f_{bp}$  = Concrete stress at the bottom fiber of the girder due to prestressing, ksi
- $P_{se}$  = Effective pretension force after all losses, kips
- $A$  = Girder cross-sectional area, in.<sup>2</sup>
- $e_c$  = Eccentricity of strand group at the midspan, in.
- $S_b$  = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder, in.<sup>3</sup>

#### 4.4.4.4 Prestress Losses

**4.4.4.4.1 General.** The losses in the prestressing force occur over time due to various reasons resulting in a reduced prestressing force. The prestress losses can be categorized as immediate losses and time dependent losses. The prestress loss due to initial steel relaxation and elastic shortening are grouped into immediate losses. The prestress loss due to concrete creep, concrete shrinkage and steel relaxation after transfer are grouped into time dependent losses. There is an uncertainty in the prestress loss over the time as it depends on many factors which cannot be calibrated accurately. Previous research has led to empirical formulas to predict the loss of prestress that are fairly accurate. A more accurate estimate of the prestress losses can be made using the time-step method. The AASHTO Standard and LRFD Specifications recommend the use of more accurate methods, like the time-step method, for exceptionally long spans or for unusual designs. However, for the parametric study the time-step method was not used as the spans were fairly standard.

AASHTO Standard Specifications provides two options to estimate the loss of prestress. The first option is the lump sum estimate of the total loss of prestress provided by AASHTO Standard Table 9.16.2.2. The second option is to use a detailed method for estimation of prestress losses that is believed to yield a more accurate estimate of losses in prestress as compared to the lump sum estimate. The detailed method is used in the parametric study to estimate the prestress losses. The AASHTO Standard Article 9.16.2 gives the empirical formulas for the detailed estimation of prestress losses as outlined in



the following sections. These formulas are applicable when normal weight concrete and 250 ksi or 270 ksi low-relaxation strands are used.

The AASHTO LRFD Specifications specifies empirical formulas to determine the instantaneous losses. For time-dependent losses, two different options are provided. The first option is to use a lump sum estimate of time-dependent losses given by AASHTO LRFD Article 5.9.5.3. The second option is to use refined estimates of time-dependent losses given by AASHTO LRFD Article 5.9.5.4. The refined estimates outlined in the following sections are used for the parametric study as they are more accurate than the lump sum estimate. The refined estimates are not applicable for prestressed concrete girders exceeding a span length of 250 ft. or made using concrete other than normal weight concrete.

**4.4.4.4.2 Instantaneous Losses.** Instantaneous losses include the loss of prestress due to elastic shortening and initial relaxation of steel. However, the Standard Specifications do not provide the formula to estimate the initial steel relaxation. Rather, only the formula for the estimation of total steel relaxation is provided. Thus for estimating the instantaneous prestress loss for the Standard designs, half the total prestress loss due to steel relaxation is considered as the instantaneous loss and other half as the time-dependent loss. This method is recommended by the TxDOT Bridge Design Manual (TxDOT 2001).

The instantaneous prestress loss is given by the following expression.

$$\Delta f_{pi} = (ES + \frac{1}{2}CR_s) \quad (4.44)$$

The percent instantaneous loss is calculated using the following expression.

$$\% \Delta f_{pi} = \frac{100(ES + \frac{1}{2}CR_s)}{0.75f'_s} \quad (4.45)$$

where:

$$\Delta f_{pi} = \text{Instantaneous prestress loss, ksi}$$

$ES$  = Prestress loss due to elastic shortening, ksi (see Sec. 4.4.4.4.5)

$CR_S$  = Prestress loss due to steel relaxation, ksi (see Sec. 4.4.4.4.6)

$f'_s$  = Ultimate strength of prestressing strands, ksi

The LRFD Specifications provide the following expression to estimate the instantaneous loss of prestress.

$$\Delta f_{pi} = (\Delta f_{pES} + \Delta f_{pRI}) \quad (4.46)$$

The percent instantaneous loss is calculated using the following expression

$$\% \Delta f_{pi} = \frac{100(\Delta f_{pES} + \Delta f_{pRI})}{f_{pj}} \quad (4.47)$$

where:

$\Delta f_{pES}$  = Prestress loss due to elastic shortening, ksi (see Sec. 4.4.4.4.5)

$\Delta f_{pRI}$  = Prestress loss due to steel relaxation at transfer, ksi (see Sec. 4.4.4.4.6)

$f_{pj}$  = Jacking stress in prestressing strands, ksi

**4.4.4.4.3 Time-Dependent Losses.** The time dependent prestress losses include those due to concrete creep, concrete shrinkage, and steel relaxation after transfer. The time dependent loss for the Standard designs is calculated using the following expression.

$$\text{Time Dependent Loss} = SH + CR_C + 0.5(CR_S) \quad (4.48)$$

where:

$SH$  = Prestress loss due to concrete shrinkage, ksi (see Sec. 4.4.4.4.8)

$CR_C$  = Prestress loss due to concrete creep, ksi (see Sec. 4.4.4.4.7)

$CR_S$  = Prestress loss due to steel relaxation, ksi (see Sec. 4.4.4.4.6)

The following expression is used to estimate the time-dependent losses for designs based on the LRFD Specifications.

$$\text{Time Dependent Loss} = \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2} \quad (4.49)$$

where:

$\Delta f_{pSR}$  = Prestress loss due to concrete shrinkage, ksi (see Sec. 4.4.4.4.8)

$\Delta f_{pCR}$  = Prestress loss due to concrete creep, ksi (see Sec. 4.4.4.4.7)

$\Delta f_{pR2}$  = Prestress loss due to steel relaxation after transfer, ksi (see Sec. 4.4.4.4.6)

**4.4.4.4.4 Total Prestress Loss.** The total loss and percent total loss of prestress is calculated using the following expressions.

For designs based on the Standard Specifications:

$$\Delta f_{pT} = ES + SH + CR_C + CR_S \quad (4.50)$$

$$\% \Delta f_{pT} = \frac{100(ES + SH + CR_C + CR_S)}{0.75f'_s} \quad (4.51)$$

For designs based on the LRFD Specifications:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pRI} + \Delta f_{pR2} \quad (4.52)$$

$$\% \Delta f_{pT} = \frac{100(\Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pRI} + \Delta f_{pR2})}{f_{pj}} \quad (4.53)$$

where:

$\Delta f_{pT}$  = Total prestress loss, ksi

$f_{pj}$  = Jacking stress in prestressing strands, ksi

**4.4.4.4.5 Elastic Shortening.** The AASHTO Standard Specifications specify the following expression to estimate the prestress loss in pretensioned members due to elastic shortening ( $ES$ ).

$$ES = \frac{E_s}{E_{ci}} f_{cir} \quad (4.54)$$

where:

- $E_s$  = Modulus of elasticity of prestressing strands, ksi
- $E_{ci}$  = Modulus of elasticity of girder concrete at transfer, ksi
 
$$= 33,000(w_c)^{3/2} \sqrt{f'_{ci}}$$
- $w_c$  = Unit weight of girder concrete, kcf
- $f'_{ci}$  = Girder concrete strength at transfer, ksi
- $f_{cir}$  = Average concrete stress at the center-of-gravity of the pretensioning steel due to pretensioning force and dead load of girder immediately after transfer, ksi
 
$$= \frac{P_{si}}{A} + \frac{P_{si}e_c^2}{I} - \frac{(M_g)e_c}{I}$$
- $P_{si}$  = Pretensioning force after allowing for the initial prestress losses, kips
- $M_g$  = Unfactored bending moment due to girder self weight, k-in.
- $e_c$  = Eccentricity of the prestressing strands at the midspan, in.
- $A$  = Area of cross-section of the girder, in.<sup>2</sup>
- $I$  = Moment of inertia of the girder section, in.<sup>4</sup>

The AASHTO LRFD Specifications specify a similar expression to determine the loss in prestress due to elastic shortening ( $\Delta f_{pES}$ ).

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \quad (4.55)$$

where:

$E_p$  = Modulus of elasticity of prestressing reinforcement, ksi

$E_{ci}$  = Modulus of elasticity of girder concrete at release, ksi

$$= 33,000(w_c)^{3/2} \sqrt{f'_{ci}}$$

$w_c$  = Unit weight of girder concrete, kcf

$f'_{ci}$  = Girder concrete strength at transfer, ksi

$f_{cgp}$  = Sum of concrete stresses at the center-of-gravity of the prestressing steel due to prestressing force at transfer and self weight of the member at sections of maximum moment, ksi

$$= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g) e_c}{I}$$

$P_i$  = Pretension force after allowing for the initial prestress losses, kips

$M_g$  = Unfactored bending moment due to girder self weight, k-in.

$e_c$  = Eccentricity of the prestressing strand group at the midspan, in.

$A$  = Area of girder cross-section, in.<sup>2</sup>

$I$  = Moment of inertia of the girder section, in.<sup>4</sup>

**4.4.4.4.6 Steel Relaxation.** The AASHTO Standard Specifications provide the following expression to estimate the loss of prestress due to steel relaxation ( $CR_S$ ).

$$CR_S = 5000 - 0.10 ES - 0.05(SH + CR_C) \quad (4.56)$$

where:

$ES$  = Prestress loss due to elastic shortening, ksi (see Sec. 4.4.4.4.5)

$SH$  = Prestress loss due to concrete shrinkage, ksi (see Sec. 4.4.4.4.8)

$CR_C$  = Prestress loss due to concrete creep, ksi (see Sec. 4.4.4.4.7)

The AASHTO LRFD Specifications provide the following expressions to estimate the prestress losses due to relaxation of steel.

At transfer - low-relaxation strands initially stressed in excess of  $0.5f_{pu}$ :

$$\Delta f_{pRI} = \frac{\log(24.0t)}{40} \left[ \frac{f_{pj}}{f_{py}} - 0.55 \right] f_{pj} \quad (4.57)$$

where:

- $\Delta f_{pRI}$  = Prestress loss due to steel relaxation at transfer, ksi
- $t$  = Time estimated in days from stressing to transfer [taken as 1 day for this study consistent with the TxDOT bridge design software PSTRS 14, (TxDOT 2005)]
- $f_{pj}$  = Initial stress in tendon at the end of stressing, ksi
- $f_{py}$  = Specified yield strength of prestressing steel, ksi

After transfer - for low-relaxation strands:

$$\Delta f_{pR2} = 0.3 [20.0 - 0.4 \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})] \quad (4.58)$$

where:

- $\Delta f_{pR2}$  = Prestress loss due to steel relaxation after transfer, ksi
- $\Delta f_{pES}$  = Prestress loss due to elastic shortening, ksi
- $\Delta f_{pSR}$  = Prestress loss due to concrete shrinkage, ksi
- $\Delta f_{pCR}$  = Prestress loss due to concrete creep, ksi

**4.4.4.4.7 Concrete Creep.** The Standard Specifications provide the following expression to estimate the prestress loss due to concrete creep ( $CR_C$ ):

$$CR_C = 12 f_{cir} - 7 f_{cds} \quad (4.59)$$

where:

- $f_{cir}$  = Average concrete stress at the center-of-gravity of the pretensioning steel due to pretensioning force and dead load of girder immediately after transfer, ksi (see Sec. 4.4.4.4.5)

$$\begin{aligned}
f_{cds} &= \text{Concrete stress at the center-of-gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied, ksi} \\
&= \frac{M_S e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c} \\
M_S &= \text{Moment due to slab weight, k-in.} \\
M_{SDL} &= \text{Superimposed dead load moment, k-in.} \\
e_c &= \text{Eccentricity of the strand at the midspan, in.} \\
y_{bc} &= \text{Distance from the centroid of the composite section to extreme bottom fiber of the precast girder, in.} \\
y_{bs} &= \text{Distance from center-of-gravity of the strands at midspan to the bottom of the girder, in.} \\
I &= \text{Moment of inertia of the non-composite section, in.}^4 \\
I_c &= \text{Moment of inertia of composite section, in.}^4
\end{aligned}$$

The LRFD Specifications provide a similar expression as Standard Specifications to estimate the loss of prestress due to creep of concrete ( $\Delta f_{pCR}$ ).

$$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp} \geq 0 \quad (4.60)$$

where:

$$\begin{aligned}
f_{cgp} &= \text{Sum of concrete stresses at the center-of-gravity of the prestressing steel due to prestressing force at transfer and self weight of the member at sections of maximum moment, ksi (see Sec. 4.4.4.4.5)} \\
\Delta f_{cdp} &= \text{Change in concrete stresses at the center-of-gravity of the prestressing steel due to permanent loads except the dead load present at the time the prestress force is applied calculated at the same section as } f_{cgp}, \text{ ksi} \\
&= \frac{M_S e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}
\end{aligned}$$

The additional variables are defined for Equation 4.59.

**4.4.4.4.8 Concrete Shrinkage.** The Standard Specifications provide the following expression to estimate the loss in prestressing force due to concrete shrinkage ( $SH$ ).

$$SH = 17,000 - 150 RH \quad (4.61)$$

where:

$RH$  = Mean annual ambient relative humidity in percent, taken as 60 percent for this parametric study.

The LRFD Specifications specify a similar expression to estimate the loss of prestress due to concrete shrinkage ( $\Delta f_{pSR}$ ).

$$\Delta f_{pSR} = 17 - 0.15 H \quad (4.62)$$

where:

$H$  = Mean annual ambient relative humidity in percent, taken as 60 percent for this parametric study.

#### *4.4.4.5 Final Estimate of Required Prestress and Concrete Strengths*

The TxDOT methodology is used to optimize the number of strands and the concrete strengths at release and service. This methodology involves several iterations of updating the prestressing strands and concrete strengths to satisfy the allowable stress limits. The step by step methodology is described as follows.

1. An initial estimate of the concrete strengths is taken as 4000 psi at release and 5000 psi at service. The prestress losses are calculated for the estimated preliminary number of strands using the estimated concrete strengths.
2. The calculation of prestress loss due to elastic shortening depends on the initial prestressing force. As the initial loss is unknown at the beginning of the prestress loss calculation process, an initial loss of eight percent is assumed. Based on this assumption, the prestress loss due to elastic



shortening, concrete creep, concrete shrinkage, and steel relaxation at transfer and at service are calculated.

The initial loss percentage is computed. If the initial loss percentage is different from eight percent, a second iteration is made using the obtained initial loss percentage from the previous iteration. The process is repeated until the initial loss percent converges to 0.1 percent of the previous iteration. The effective prestress at transfer and at service are calculated using the following expressions.

For Standard Specifications:

$$f_{si} = 0.75 f'_s - \Delta f_{pi} \quad (4.63)$$

$$f_{se} = 0.75 f'_s - \Delta f_{pT} \quad (4.64)$$

where:

- $f_{si}$  = Effective initial prestress, ksi
- $f_{se}$  = Effective final prestress, ksi
- $f'_s$  = Ultimate strength of prestressing strands, ksi
- $\Delta f_{pi}$  = Instantaneous prestress losses, ksi
- $\Delta f_{pT}$  = Total prestress losses, ksi

For LRFD Specifications:

$$f_{pi} = f_{pj} - \Delta f_{pi} \quad (4.65)$$

$$f_{pe} = f_{pj} - \Delta f_{pT} \quad (4.66)$$

where:

- $f_{pi}$  = Effective initial prestress, ksi
- $f_{pe}$  = Effective final prestress, ksi
- $f_{pj}$  = Jacking stress in prestressing strands, ksi

The total effective prestressing force is calculated by multiplying the calculated effective prestress per strand, area of strand, and the number of strands. The concrete stress at the bottom fiber of the girder due to the effective prestressing force is calculated using Equation 4.43. If this stress is found to be less than the required prestress, the number of strands is incremented by two in each step until the required prestress is achieved. The initial bottom fiber stress at the hold-down points is calculated and using the allowable stress limit at this section, the required concrete strength at release is determined. The number of strands and concrete strength at release is used to determine the prestress losses for the next trial. The effective prestress after the losses at transfer and at service are then calculated.

The initial concrete stresses at the top and bottom fibers at the girder end, transfer length section, and hold-down points are determined using the effective prestress at transfer. The final concrete stresses at the top and bottom fibers at the midspan section are determined using the applied loads and effective prestress at service. The initial tensile stress at the top fiber at the girder end is minimized by harping the web strands at the girder end. The web strands are incrementally raised as a unit by two inches in each step. The steps are repeated until the top fiber stress satisfies the allowable stress limit or the centroid of the topmost row of the harped strands is at a distance of two inches from the top fiber of the girder. If the later case is applicable, the concrete strength at release is updated based on the governing stress.

The expressions used for the determination of stresses at each location are outlined in the following section. The concrete stress at each location is compared with the allowable stresses and if necessary, corresponding concrete strength is updated. This process is repeated until the concrete strengths at release and at service converges within 10 psi of the values calculated in the previous iteration. The governing concrete strength at release and at service is established using the greatest required concrete strengths. The program terminates if the required concrete strength at release or service exceeds predefined maximum values for Standard girder designs (discussed in Sec 4.5).

#### 4.4.4.6 Check for Concrete Stresses

**4.4.4.6.1 General.** The expressions used to calculate the concrete stress at different sections is outlined in the following subsections. These expressions utilize the notation for the LRFD Specifications. The same expressions are used for calculating the stresses for designs following the Standard Specifications with the corresponding notation. The calculated concrete stress is compared with the corresponding allowable stress limit provided in Table 4.4.

**4.4.4.6.2 Concrete Stress at Transfer.** The concrete stress at transfer at different locations along the girder length is determined using the following expressions.

At girder ends - top fiber:

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_e}{S_t} \quad (4.67)$$

At girder ends - bottom fiber:

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_e}{S_b} \quad (4.68)$$

At transfer length section - top fiber:

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_t}{S_t} + \frac{M_g}{S_t} \quad (4.69)$$

At transfer length section - bottom fiber:

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_t}{S_b} - \frac{M_g}{S_b} \quad (4.70)$$

At hold-down points – top fiber:

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{M_g}{S_t} \quad (4.71)$$

At hold-down points – bottom fiber:

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{M_g}{S_b} \quad (4.72)$$

At midspan – top fiber:

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{M_g}{S_t} \quad (4.73)$$

At midspan – bottom fiber:

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{M_g}{S_b} \quad (4.74)$$

where:

- $f_{ti}$  = Initial concrete stress at the top fiber of the girder, ksi
- $f_{bi}$  = Initial concrete stress at the bottom fiber of the girder, ksi
- $P_i$  = Pretension force after allowing for the initial losses, kips
- $M_g$  = Unfactored bending moment due to girder self weight at the location under consideration, k-in.
- $e_c$  = Eccentricity of the strands at the midspan and hold-down point, in.
- $e_e$  = Eccentricity of the strands at the girder ends, in.
- $e_t$  = Eccentricity of the strands at the transfer length section, in.
- $A$  = Area of girder cross-section, in.<sup>2</sup>
- $S_b$  = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder, in.<sup>3</sup>
- $S_t$  = Section modulus referenced to the extreme top fiber of the non-composite precast girder, in.<sup>3</sup>

**4.4.4.6.3 Concrete Stress at Intermediate Stage.** The concrete stress at the midspan for the intermediate load stage is determined using the following expressions.

$$f_t = \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}} \quad (4.75)$$

$$f_b = \frac{P_{se}}{A} + \frac{P_{se} e_c}{S_b} - \frac{M_g + M_S}{S_b} - \frac{M_{SDL}}{S_{bc}} \quad (4.76)$$

where:

$f_t$  = Concrete stress at the top fiber of the girder, ksi

$f_b$  = Concrete stress at the bottom fiber of the girder, ksi

$P_{se}$  = Effective pretension force after all losses, kips

$M_S$  = Bending moment due to slab weight, k-in.

$M_{SDL}$  = Bending moment due to superimposed dead load, k-in.

$S_{tg}$  = Composite section modulus referenced to the extreme top fiber of the precast girder, in.<sup>3</sup>

$S_{bc}$  = Composite section modulus referenced to the extreme bottom fiber of the precast girder, in.<sup>3</sup>

The additional variables are the same as defined for Equations 4.67 to 4.74.

**4.4.4.6.4 Concrete Stresses at Service.** The concrete stress at service at the midspan for different load combinations is determined using the following expressions. For the Standard Specifications, the stresses for the following cases of the Service I load combination were investigated.

Concrete stress at top fiber of the girder under:

Case (I): Live load + 0.5 × (pretensioning force + dead loads)

$$f_t = \frac{M_{LL+I}}{S_{tg}} + 0.5 \left( \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}} \right) \quad (4.77)$$

Case (II): Service loads

$$f_t = \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL} + M_{LL+I}}{S_{tg}} \quad (4.78)$$

Concrete stresses at bottom fiber of the girder under service loads:

$$f_b = \frac{P_{se}}{A} + \frac{P_{se} e_c}{S_b} - \frac{M_g + M_S}{S_b} - \frac{M_{SDL} + M_{LL+I}}{S_{bc}} \quad (4.79)$$

where:

$M_{LL+I}$  = Moment due to live load including impact at the midspan, k-in.

The additional variables are the same as defined for Equation 4.76.

For the LRFD Specifications, the stresses for the Service I and Service III load combinations were investigated.

Concrete stresses at top fiber of the girder under:

Service I - Case (I):  $0.5 \times$  (effective prestress force + permanent loads) + transient loads

$$f_t = 0.5 \left( \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}} \right) + \frac{M_{LL} + M_{LT}}{S_{tg}} \quad (4.80)$$

Service I - Case (II): Permanent and transient loads

$$f_t = \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}} + \frac{M_{LL} + M_{LT}}{S_{tg}} \quad (4.81)$$

Service III: Concrete stresses at bottom fiber of the girder

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe} e_c}{S_b} - \frac{M_g + M_S}{S_b} - \frac{M_{SDL} + 0.8(M_{LT} + M_{LL})}{S_{bc}} \quad (4.82)$$

where:

$P_{pe}$  = Effective pretension force after all losses, kips

$M_{LT}$  = Bending moment due to truck load including impact, at the section, k-in.

$M_{LL}$  = Bending moment due to lane load at the section, k-in.

#### 4.4.4.7 Allowable Stress Limits

The allowable stress limits specified by the Standard and LRFD Specifications are presented in this section. The  $f'_c$  and  $f'_{ci}$  values are expressed in psi units for calculating the allowable stresses based on the Standard Specifications, whereas ksi units are used for the LRFD Specifications.

**Table 4.4. Allowable Stress Limits Specified by AASHTO Standard and LRFD Specifications.**

| Load Stage  | Type of Stress                      | Allowable Stresses         |                             |
|---|-------------------------------------|----------------------------|-----------------------------|
|   |                                     | Standard (psi)             | LRFD (ksi)                  |
| Transfer Stage: Stresses immediately after transfer   | Compression                         | $0.6 f'_{ci}$              | $0.6 f'_{ci}$               |
|   | Tension                             | $7.5 \sqrt{f'_{ci}}^{(1)}$ | $0.24 \sqrt{f'_{ci}}^{(2)}$ |
| Intermediate Stage: After CIP concrete slab hardens. Stresses due to effective prestress and permanent loads only | Compression                         | $0.40 f'_c$                | $0.45 f'_c$                 |
|   | Tension                             | $6 \sqrt{f'_c}$            | $0.19 \sqrt{f'_c}$          |
| Final Stage: Stresses at service  | Compression: Case I <sup>(3)</sup>  | $0.60 f'_c$                | $0.6 \phi_w f'_c{}^{(4)}$   |
|   | Compression: Case II <sup>(3)</sup> | $0.40 f'_c$                | $0.40 f'_c$                 |
|   | Tension                             | $6 \sqrt{f'_c}$            | $0.19 \sqrt{f'_c}$          |

Notes:

1. The specified limit is the maximum allowable tensile stress at transfer. However, if the calculated tensile stress exceeds 200 psi or  $3 \sqrt{f'_{ci}}$  whichever is smaller, bonded reinforcement should be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section.

2. The specified limit is the maximum allowable tensile stress at transfer. To use this limit bonded reinforcement shall be provided which is sufficient to resist the tension force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of  $0.5f_y$ , not to exceed 30 ksi. If the stresses does not exceed smaller of  $0.0948\sqrt{f'_{ci}}$  and 0.200 ksi bonded reinforcement is not required.
3. Case (I): For all load combinations  
Case (II): For live load +  $0.5 \times$  (effective pretension force + dead loads)
4. AASHTO LRFD Article 5.9.4.2 specifies the reduction factor  $\phi_w$  to be taken as 1.0 when the web and flange slenderness ratios are not greater than 15. If the slenderness ratio of either the web or the flange exceeds 15, LRFD Article 5.7.4.7.1 shall be used to compute  $\phi_w$ .

The allowable stress limits for low-relaxation prestressing strands for AASHTO Standard and LRFD Specifications are provided in Table 4.5

**Table 4.5. Stress Limits for Low-Relaxation Prestressing Strands Specified by the AASHTO Standard and LRFD Specifications.**

| Condition   | Stress Limit  |
|---|---------------|
| Immediately Prior to Transfer<br>(after Initial losses) | $0.75 f_{pu}$ |
| Service limit state after all losses                    | $0.80 f_{py}$ |

where:

- $f_{pu}$  = Specified tensile strength of prestressing steel, ksi  
 $f_{py}$  = Yield strength of prestressing steel, ksi

#### 4.4.4.8 Summary of Changes in Service Load Design

The major differences between the AASHTO Standard Specifications and AASHTO LRFD Specifications in the service load design are summarized in Table 4.6. Additional details are provided in the previous subsections.



**Table 4.6. Significant Differences between Design Provisions for I-Shaped Prestressed Concrete Bridge Girders.**

| Description                 | Standard Specifications   | LRFD Specifications  |
|-----------------------------|---|--|
| Live Load                   | 1. Standard HS 20-44 truck loading<br>2. HS 20-44 lane loading<br>3. Tandem loading<br>Whichever produces maximum stresses  | 1. HS 20-44 truck and uniform lane loading (HL-93)<br>2. Tandem and uniform lane loading<br><br>Whichever combination produces maximum stresses  |
| Impact Load                 | $I = \frac{50}{L + 125} \leq 30\%$  | 33 percent of the live load  |
| Moment Distribution Factors | $S/11$ , where $S$ is the spacing between the girders   | For two or more lanes loaded:<br>$DFM = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12.0L t_s^3}\right)^{0.1}$<br>For one design lane loaded:<br>$DFM = 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12.0L t_s^3}\right)^{0.1}$                             |
| Shear Distribution Factors  | $S/11$<br>where $S$ is the spacing between the girders  | For two or more lanes loaded:<br>$DFV = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^2$<br>For one design lane loaded:<br>$DFV = 0.36 + \left(\frac{S}{25.0}\right)$  |
| Load Combinations           | For service load design<br>$1.00D + 1.00(L+I)$<br><br>For load factor design<br>$1.3[1.00D + 1.67(L+I)]$  | Service I: $Q = 1.00(DC + DW) + 1.00(LL + IM)$<br>Service III: $Q = 1.00(DC + DW) + 0.80(LL + IM)$<br>Strength I:<br>Max. $Q = 1.25(DC) + 1.50(DW) + 1.75(LL + IM)$<br>Min. $Q = 0.90(DC) + 0.65(DW) + 1.75(LL + IM)$  |
| Prestress Losses            | Instantaneous loss<br>$(ES + 0.5CR_s)$<br><br>Steel relaxation loss<br>$CR_s = 5000 - 0.10 ES - 0.05(SH + CR_C)$<br><br>Total prestress loss<br>$ES + SH + CR_C + CR_s$ | Instantaneous loss<br>$\Delta f_{pi} = (\Delta f_{pES} + \Delta f_{pR1})$<br><br>Initial steel relaxation loss<br>$\Delta f_{pR1} = \frac{\log(24.0t)}{40} \left[ \frac{f_{pj}}{f_{py}} - 0.55 \right] f_{pj}$<br><br>Total prestress loss<br>$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$ |

#### 4.4.5 Fatigue Limit State

The AASHTO LRFD Specifications require that the fatigue in the prestressing strands be checked. This limit state was not provided in AASHTO Standard Specifications. LRFD Specifications Article 5.5.3 specifies that the check for fatigue of the prestressing strands is not necessary for fully prestressed components that are designed to have extreme fiber tensile stress due to Service III limit state within the specified limit of  $0.19\sqrt{f'_c}$  (same as  $6\sqrt{f'_c}$  psi). In the parametric study, the girders are designed to always satisfy this specified limit. Hence the Fatigue limit state check is not required.

#### 4.4.6 Flexural Strength Limit State

##### 4.4.6.1 General

The flexural strength limit state design requires the reduced nominal moment capacity of the member to be greater than the factored ultimate design moment, expressed as follows.

$$\phi M_n \geq M_u \quad (4.83)$$

where:

- $M_u$  = Factored ultimate moment at a section, k-ft.
- $M_n$  = Nominal moment strength of a section, k-ft.
- $\phi$  = Resistance factor = 1.0 for flexure and tension of prestressed concrete members (AASHTO Standard and LRFD Specifications)

The total bending moment for the ultimate limit state according to AASHTO Standard Specifications given by the Group I factored load combination is as follows.

$$M_u = 1.3[M_g + M_S + M_{SDL} + 1.67(M_{LL+I})] \quad (4.84)$$

where:

- $M_g$  = Unfactored bending moment due to girder self-weight, k-ft.
- $M_S$  = Unfactored bending moment due to slab weight, k-ft.
- $M_{SDL}$  = Unfactored bending moment due to superimposed dead (barrier and asphalt wearing surface) load, k-ft.
- $M_{LL+I}$  = Bending moment due to live load including impact, k-ft.

The total ultimate bending moment for Strength I limit state, according to the AASHTO LRFD Specifications is as follows.

$$M_u = 1.25(DC) + 1.5(DW) + 1.75(LL + IM) \quad (4.85)$$

where:

- $DC$  = Bending moment due to all dead loads except wearing surface, k-ft.
- $DW$  = Bending moment due to wearing surface load, k-ft.
- $LL+IM$  = Bending moment due to live load and impact, k-ft.

The flexural strength limit state design reduces to a check as the number of prestressing strands and the concrete strengths are already established from the service load design. For the case when the flexural limit state is not satisfied, the number of strands is incremented by two, and the service load stresses are checked and concrete strengths are updated if required. This process is carried out until the flexural limit state is satisfied. However, for prestressed concrete members, service load design almost always governs, and the designs satisfying service load criteria usually satisfy the flexural limit state.

#### 4.4.6.2 Assumptions for Strength Limit State Design

The AASHTO Standard and LRFD Specifications use different approaches for the calculation of the flexural moment resistance of the prestressed concrete girder. The two specifications essentially follow the force and equilibrium formulations with slight

modifications. Because the depth of neutral axis and the effective prestress are inter-related, modifications in the force equilibrium formulations are required. The Standard Specifications uses an empirical formulation of effective prestress, and the depth of neutral axis is calculated using this value.

The LRFD Specifications uses the ultimate strength of the prestressing strands to establish the depth of neutral axis based on which the effective prestress is calculated. The differences in the methodologies followed by the Standard and LRFD Specifications are outlined in this section. The methodology used in the parametric study for designs based on Standard and LRFD Specifications is also presented. The Standard and LRFD Specifications also allow the use of strut and tie model to determine the design moment strength of the prestressed concrete girders. However, this approach is not considered for the parametric study.

The AASHTO Standard and LRFD Specifications provide the formula for the moment resistance of prestressed concrete girders assuming:

1. The members are uncracked,
2. The maximum usable strain in unconfined concrete at extreme compression fiber is not greater than 0.003,
3. The tensile strength of concrete is neglected,
4. A rectangular stress distribution in the concrete compression zone,
5. A linear variation of strain over the section depth, and
6. The section is transformed based on actual concrete strengths of the slab and the girder.

In the present parametric study, the last assumption will not be used. A more accurate estimate of the compression contribution of each element (CIP slab, girder flange and girder web) will be evaluated for flanged section behavior. This requires the Standard and LRFD expressions to be modified. The modified expressions are provided in the following section.

The Standard and LRFD define rectangular and flanged section behavior in different ways. The Standard Specifications Article 9.17.2 specifies that rectangular or

flanged sections, having prestressing steel only can be considered to behave like rectangular sections if the depth of equivalent rectangular stress block,  $a$ , is less than the thickness of the compression flange (slab). The LRFD Specifications considers the section to behave like a rectangular section if the depth of neutral axis,  $c$ , lies within the flange (slab).

#### 4.4.6.3 Equivalent Rectangular Stress Block

The stress distribution in the compression concrete is approximated with an equivalent rectangular stress distribution of intensity  $0.85f'_c$  over a zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at the distance  $a = \beta_1 c$ , where  $\beta_1$  is the stress block factor. The value of  $\beta_1$  is 0.85 for concrete strengths less than 4.0 ksi and is reduced at a rate of 0.05 for each 1.0 ksi of strength in excess of 4.0 ksi, but is not taken less than 0.65. For flanged section behavior, the concrete strengths are different for the flange (slab) and the web (girder) which brings an inconsistency in the calculation of the parameter  $\beta_1$  if the section is not transformed. The LRFD Specifications Article 5.7.2.2 provides three different options to evaluate  $\beta_1$  when the cross-section behaves as a flanged section:

1. Use the  $\beta_1$  value of the slab for composite design,
2. Use the actual values of  $\beta_1$  for each section, or
3. Use the average value of  $\beta_1$  given by the following expression.

$$\beta_{1avg} = \frac{\Sigma(f'_c A_{cc} \beta_1)}{\Sigma(f'_c A_{cc})} \quad (4.86)$$

where:

$\beta_1$  = Stress block factor

$f'_c$  = Concrete strength at service, ksi

$A_{cc}$  = Area of concrete element in compression with the corresponding concrete strength, in.<sup>2</sup>

The average value of  $\beta_1$  given by the above equation was used for the parametric study for designs based on the LRFD Specifications to determine the depth of neutral axis, when the section was found to be behaving as a flanged section. The  $\beta_1$  for the slab concrete is used in the evaluation of effective stress in the prestressing steel for the Standard Specifications.

#### 4.4.6.4 Effective Stress in Prestressing Steel at Nominal Flexural Resistance

The AASHTO Standard Specifications provides the following empirical relation to estimate the average stress in the bonded prestressing steel at ultimate load.

$$f_{su}^* = f_s' \left( 1 - \frac{\gamma^*}{\beta_1} \rho^* \frac{f_s'}{f_c'} \right) \quad (4.87)$$

where:

- $f_{su}^*$  = Average stress in pretensioning steel at ultimate load, ksi
- $\gamma^*$  = Factor for type of prestressing steel, taken as 0.28 for low-relaxation strand
- $\beta_1$  = Stress block factor
- $\rho^*$  = Ratio of prestressing steel
- $= \frac{A_s^*}{bd}$
- $A_s^*$  = Area of pretensioned reinforcement, in.<sup>2</sup>
- $b$  = Effective flange width, in.
- $d$  = Distance from top of slab to centroid of prestressing strands, in.
- $f_s'$  = Ultimate strength of prestressing strands, ksi
- $f_c'$  = Concrete strength at service (taken as  $f_c'$  of slab to be conservative), ksi

Equation 4.87 is applicable when the effective prestress after losses is not less than  $0.5 f_s'$ .

The LRFD Specifications specify the following expression to determine the stress in prestressing steel. This expression is applicable when the effective prestress after losses,  $f_{pe}$  is not less than  $0.5 f_{pu}$ , where  $f_{pu}$  is the ultimate strength of the prestressing strands.

$$f_{ps} = f_{pu} \left( 1 - k \frac{c}{d_p} \right) \quad (4.88)$$

where:

$f_{ps}$  = Average stress in prestressing steel, ksi

$f_{pu}$  = Specified tensile strength of prestressing steel, ksi

$k$  =  $2 \left( 1.04 - \frac{f_{py}}{f_{pu}} \right) = 0.28$  for low-relaxation strands

$c$  = Distance between neutral axis and the compressive face, in.

$d_p$  = Distance from extreme compression fiber to the centroid of the prestressing tendons, in.

#### 4.4.6.5 Depth of Neutral Axis and Design Flexural Moment Resistance

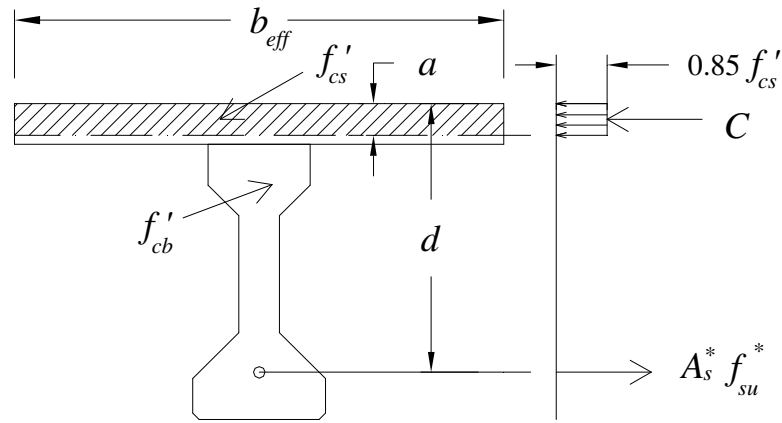
The provisions of the AASHTO Standard and LRFD Specifications for calculating the depth of neutral axis and the design moment resistance of the section are outlined below. The methodology used in the parametric study is also described.

**4.4.6.5.1 Standard Specifications.** The flexural behavior of the section at ultimate conditions is classified as rectangular or flanged based on the depth of equivalent rectangular stress block. The AASHTO Standard Specifications specify that if the condition in Equation 4.89 is satisfied, the section behavior shall be considered as rectangular. Figure 4.6 illustrates the rectangular section behavior of the composite section and the corresponding stress distribution.

$$a = \frac{A_s^* f_{su}^*}{0.85 f_{cs}' b} \leq t \quad (4.89)$$

where:

- $a$  = Depth of equivalent stress block, in.
- $A_s^*$  = Area of pretensioned reinforcement, in.<sup>2</sup>
- $f_{su}^*$  = Average stress in pretensioning steel at ultimate load, ksi
- $b$  = Effective flange width (denoted as  $b_{eff}$  in Figure 4.6), in.
- $f_{cs}'$  = Flange (slab) concrete compressive strength at service, ksi
- $t$  = Depth of compression flange (slab), in.



**Figure 4.6. Rectangular Section Behavior – Standard Notation**

Expression 4.89 can be verified by simple mechanics and it is valid for the parametric study. The depth of neutral axis,  $c$ , is calculated as  $c = a/\beta_1$ , where  $\beta_1$  is calculated using the slab concrete compressive strength. The design flexural moment strength for rectangular section behavior can be evaluated using the following expression

$$\phi M_n = \phi (A_s^*) (f_{su}^*) (d) \left( 1 - 0.6 \frac{\rho^* f_{su}^*}{f_{cs}'} \right) \quad (4.90)$$



where:

$\phi$  = Strength reduction factor specified as 1.0 for prestressed concrete members.

$M_n$  = Nominal moment strength at the section, k-ft.

Additional variables are the same as defined for Equations 4.87 and 4.89.

If the condition in Equation 4.89 is not satisfied, the section shall be checked for the flanged section behavior provided by the following expression.

$$\frac{A_{sr} f_{su}^*}{0.85 f'_{cs} b'} > t \quad (4.91)$$

where:

$A_{sr} = A_s^* - A_{sf} \text{ (in.}^2\text{)}$

$A_s^* = \text{Area of pretensioned reinforcement, in.}^2$

$A_{sf} = \text{Steel area required to develop the ultimate compressive strength of the overhanging portions of the flange, in.}^2$

$A_{sf} = 0.85 f'_{cs} (b - b') t / f_{su}^*$

$b' = \text{Width of the web, in.}$

$f_{su}^* = \text{Average stress in pretensioning steel at ultimate load, ksi}$

$b = \text{Effective flange width (denoted as } b_{eff} \text{ in Figure 4.6), in.}$

$f'_{cs} = \text{Flange (slab) concrete strength at service, ksi}$

$t = \text{Depth of compression flange (slab), in.}$

The design flexural strength of the flanged section is determined as follows.

$$\phi M_n = \phi \left\{ A_{sr} f_{su}^* d \left( 1 - 0.6 \frac{A_{sr} f_{su}^*}{b' d f'_{cs}} \right) + 0.85 f'_{cs} (b - b') t (d - 0.5t) \right\} \quad (4.92)$$



$$T = A_s^* f_{su}^* \quad (4.95)$$

From equilibrium,

$$T = C_1 + C_2 \quad (4.96)$$

$$a = \frac{A_s^* f_{su}^* - 0.85 f'_{cs} b_{eff} t + 0.85 f'_{cb} b_f t}{0.85 f'_{cb} b_f} \leq (t + t_f) \quad (4.97)$$

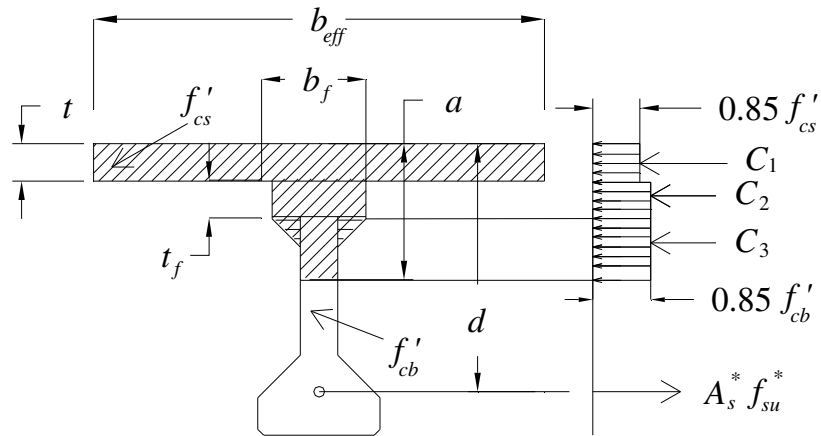
Taking moments about  $C_1$ , the nominal design flexural strength is the following.

$$\phi M_n = \phi [T(d - 0.5t) - C_2(0.5a)] \quad (4.98)$$

where:

- $T$  = Tensile force in the prestressing strands, kips
- $C_1$  = Compression force in the slab, kips
- $C_2$  = Compression force in the girder flange, kips
- $f'_{cs}$  = Flange (slab) concrete strength at service, ksi
- $f'_{cb}$  = Girder concrete strength at service, ksi
- $b_{eff}$  = Effective flange (slab) width, in.
- $b_f$  = Flange width of the girder, in.
- $t$  = Thickness of the deck slab, in.

Case II considers the lower portion of the equivalent rectangular stress block lies in the web of the girder as shown in Figure 4.8. The contribution of the fillet area is neglected for simplicity. Because the fillet area is small, the fillets do not contribute significantly to the compression force or nominal moment strength.



**Figure 4.8. Rectangular Stress Block in the Girder Web**

From Figure 4.8, the following values may be computed.

$$C_1 = 0.85 f'_{cs} b_{eff} t \quad (4.99)$$

$$C_2 = 0.85 f'_{cb} b_f t_f \quad (4.100)$$

$$C_3 = 0.85 f'_{cb} b' (a - t_f - t) \quad (4.101)$$

$$T = A_s^* f_{su}^* \quad (4.102)$$

Applying equilibrium,

$$T = C_1 + C_2 + C_3 \quad (4.103)$$

$$a = \frac{A_s^* f_{su}^* - 0.85 f'_{cs} b_{eff} t - 0.85 f'_{cb} b_f t_f + 0.85 f'_{cb} b' t_f + 0.85 f'_{cb} b' t}{0.85 f'_{cb} b'} \quad (4.104)$$

Taking moments about  $C_1$ , the nominal design flexural strength is the following.

$$\phi M_n = \phi [T(d - 0.5t) - C_2(0.5t + 0.5t_f) - C_3(0.5a + 0.5t_f)] \quad (4.105)$$

where:

$T$  = Tensile force in the prestressing strands, kips

$C_1$  = Compression force in the slab, kips

- $C_2$  = Compression force in the girder flange, kips  
 $C_3$  = Compression force in girder web, kips  
 $t_f$  = Thickness of girder flange, in.  
 $b'$  = Girder web width, in.

The additional variables are the same as defined for Equations 4.93 to 4.98.

**4.4.6.5.2 Impact of Neglecting the Fillet Area on the Design Nominal Flexural Strength.** The following design example investigates the impact of ignoring the fillet area on the design nominal flexural strength. The design example is carried out assuming the required number of strands as 90 and the following values.

- $f'_{cs}$  = Flange (slab) concrete strength at service = 4.0 ksi  
 $f'_{cb}$  = Girder concrete strength at service = 6.5 ksi  
 $b_{eff}$  = Effective flange width = 72 in.  
 $b_f$  = Flange width of the girder = 20 in.  
 $t_f$  = Thickness of girder flange = 8 in.  
 $b'$  = Girder web thickness = 8 in.  
 $b_{fil}$  = Width of fillet = 6 in.  
 $t_{fil}$  = Fillet thickness = 6 in.  
 $A_s^*$  = Area of prestressing strands =  $90(0.153) = 13.77 \text{ in.}^2$

The effective stress in the steel is calculated assuming rectangular section behavior using Equation 4.90.

$$f_{su}^* = f_s' \left( 1 - \frac{\gamma^*}{\beta_1} \rho^* \frac{f_s'}{f_c'} \right)$$

where:

$$f_{su}^* = \text{Average stress in pretensioning steel at ultimate load, ksi}$$

$$\gamma^* = \text{Factor for prestressing steel type, specified as 0.28 for low-relaxation strand}$$

$$\beta_1 = \text{Stress block factor} \\ = 0.85 \text{ for slab concrete strength of 4.0 ksi}$$

$$\rho^* = \text{Ratio of prestressing steel}$$

$$= \frac{A_s^*}{bd}$$

$$A_s^* = \text{Area of pretensioned reinforcement} = 13.77 \text{ in.}^2$$

$$b = \text{Effective flange width} = 72 \text{ in.}$$

$$d = \text{Distance from top of slab to centroid of pretensioning strands, in.} \\ = h_c - y_{bs}$$

$$h_c = \text{Depth of composite section} = 62 \text{ in.}$$

$$y_{bs} = \text{Distance of center-of-gravity of the prestressing strands from the bottom fiber of the girder, in.}$$

$$= \frac{12(2+4+6) + 10(8) + 8(10) + 6(12) + 4(14)}{90} \\ + \frac{2(16+18+20+22+24+26+28+30+32+34+36+38+40)}{90}$$

$$= 12.89 \text{ in.}$$

$$d = 62 - 12.89 = 49.11 \text{ in.}$$

$$\rho^* = \frac{13.77}{72(49.11)} = 0.0039$$

$$f_s' = \text{Ultimate strength of prestressing strands} = 270 \text{ ksi}$$

$$f_{su}^* = 270 \left[ 1 - \left( \frac{0.28}{0.85} \right) (0.0039) \left( \frac{270}{4} \right) \right] = 246.59 \text{ ksi}$$

Depth of compression block assuming rectangular section behavior calculated using Equation 4.89.

$$a = \frac{A_s^* f_{su}^*}{0.85 f_{cs}' b} = \frac{13.77(246.59)}{0.85(4)(72)} = 13.87 \text{ in.} > 8.0 \text{ in.}$$

Section behaves as a flanged section, assuming the stress block is in the girder web

Case I - Ignoring the fillet contribution:

$$C_1 = 0.85 f_{cs}' b_{eff} t = 0.85(4)(72)(8) = 1958.4 \text{ kips}$$

$$C_2 = 0.85 f_{cb}' b_f t_f = 0.85(6.5)(20)(8) = 884.0 \text{ kips}$$

$$C_3 = 0.85 f_{cb}' b' (a - t_f - t) = 0.85(6.5)(8)(a - 8 - 8) = 44.2(a - 16) \text{ kips}$$

$$T = A_s^* f_{su}^* = 13.77(246.59) = 3395.5 \text{ kips}$$

From equilibrium,

$$T = C_1 + C_2 + C_3$$

$$3395.5 = 44.2(a - 16) + 1958.4 + 884.0$$

$$a = 28.5 \text{ in.}$$

$$C_3 = 0.85 f_{cb}' (a - t_f - t) b' = 0.85(6.5)(8)(28.5 - 8 - 8) = 552.5 \text{ kips}$$

Design flexural moment strength using Equation 4.105:

$$\begin{aligned} \phi M_n &= \phi [T(d - 0.5t) - C_2(0.5t + 0.5t_f) - C_3(0.5a + 0.5t_f)] \\ &= 1.0[3395.5(49.11 - 4) - 884(4 + 4) - 552.5(14.2 + 4)] \\ &= 136043 \text{ k-in.} = 11337 \text{ k-ft.} \end{aligned}$$

Case II - Considering the fillet contribution:

$$C_1 = 0.85 f_{cs}' b_{eff} t = 0.85(4)(72)(8) = 1958.4 \text{ kips}$$

$$C_2 = 0.85 f_{cb}' b_f t_f = 0.85(6.5)(20)(8) = 884.0 \text{ kips}$$

$$C_3 = 0.85 f'_{cb} b' (a - t_f - t) = 0.85(6.5)(8)(a - 8 - 8) = 44.2(a - 16) \text{ kips}$$

Assuming the stress block depth is below the fillet end (i.e.  $a > 22$  in.) makes the whole fillet area act in compression.

$$C_{fillet} = 0.85 f'_{cb} b_{fillet} t_{fillet} = 0.85(6.5)(6)(6) = 198.9 \text{ kips}$$

$$T = A_s * f_{su} * = 13.77(246.59) = 3395.5 \text{ kips}$$

Applying equilibrium,

$$T = C_1 + C_2 + C_3 + C_{fillet}$$

$$3395.5 = 44.2(a - 16) + 1958.4 + 884.0 + 198.9$$

$$a = 24 \text{ in.} > 22 \text{ in.}$$

Assumption is good, the total fillet area acts in compression.

$$C_3 = 0.85 f'_{cb} b' (a - t_f - t) = 0.85(6.5)(8)(24 - 8 - 8) = 353.6 \text{ kips}$$

Design flexural moment strength:

$$\begin{aligned} \phi M_n &= \phi [T(d - 0.5t) - C_2(0.5t + 0.5t_f) - C_3(0.5a + 0.5t_f) - C_{fillet}(0.5t + t_f + 0.33 t_{fillet})] \\ &= 1.0[3395.5(49.11 - 4) - 884.0(4 + 4) - 353.6(12 + 4) - 198.9(4 + 8 + 2)] \\ &= 137657 \text{ k-in.} = 11471 \text{ k-ft.} \end{aligned}$$

Percent difference in the design flexural strengths from Case I and Case II:

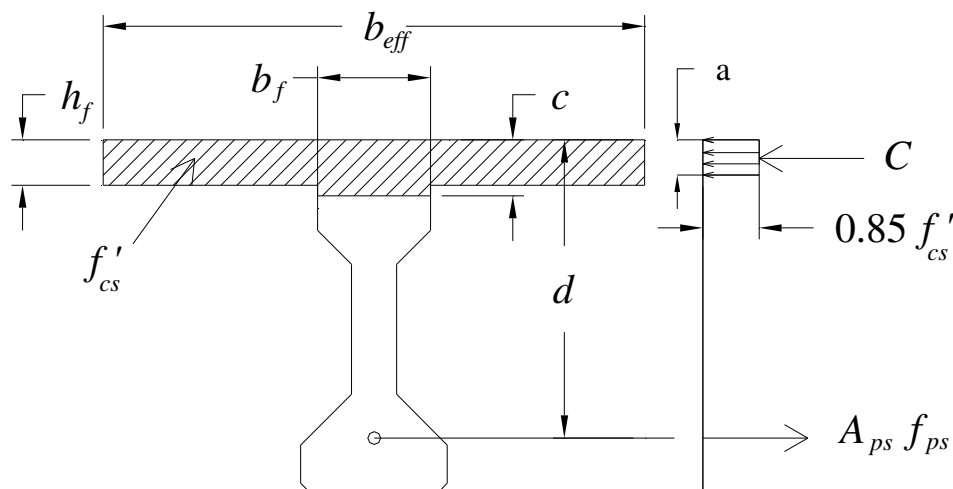
$$= 100(11471 - 11337)/11471 = \mathbf{1.18\%}$$

Thus, the fillet portion does not have a significant contribution to the design flexural strength. However, the depth of neutral axis is changed significantly when the fillet portion is ignored. If the fillet portion is considered the expression for the nominal moment strength calculation becomes much more complex which is not reasonable in

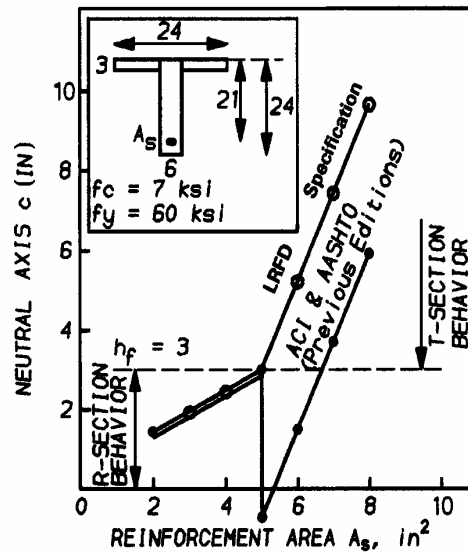


practice. Therefore, the fillet portion is ignored in the parametric study without any significant loss in accuracy.

**4.4.6.5.3 LRFD Specifications.** The LRFD Specifications assumes a section to behave as a flanged section if the neutral axis lies within the web. However, while using this assumption an inconsistency is found when the neutral axis lies within the web (i.e.  $c > h_f$ ) but the depth of the equivalent rectangular stress block is less than the flange thickness,  $h_f$  (i.e.  $a = \beta_1 c < h_f$ ). When the depth of the neutral axis,  $c$  is recomputed based on the ACI Building Code (Building Code Requirements for Structural Concrete, ACI 318R-02) approach, the value of  $c$  within the web width is observed to be smaller than  $h_f$  and even negative. This inconsistency occurs because the factor  $\beta_1$  is applied only to the web but not to the flange portion. The LRFD Specifications recommends applying the factor  $\beta_1$  for both the flange and the web portion of the girder when the section behaves as a flanged section. Figure 4.9 illustrates this case and Figure 4.10 compares the depth of the neutral axis based on the ACI approach and proposed LRFD approach.



**Figure 4.9. Neutral Axis lies in the Girder Flange and the Stress Block is in the Slab**



**Figure 4.10. Neutral Axis Depth using ACI Approach and Proposed AASHTO LRFD Approach (AASHTO LRFD Specifications 2004)**

Using either approach does not affect the value of the nominal flexural resistance significantly, but there is a significant effect on the depth of neutral axis,  $c$ . The provisions for limits for ductility requirement are based on  $c/d_e$  value, and there is a significant affect on these provisions when the proposed AASHTO LRFD Specifications approach is used. LRFD Specifications Article 5.7.3 specifies the following expressions to determine the depth of neutral axis and the design flexural moment strength.

Rectangular section behavior is assumed first to determine the depth of neutral axis, as illustrated in Figure 4.11.

$$c = \frac{A_{ps} f_{pu}}{0.85 f'_c \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}} < h_f \quad (4.106)$$

where:

$c$  = Distance between neutral axis and the compressive face, in.

$A_{ps}$  = Area of prestressing steel, in.<sup>2</sup>

$f'_{cs}$  = Compressive strength of slab concrete at service, ksi

$\beta_1$  = Stress factor of compression block (computed for  $f'_{cs}$ )

$b$  = Effective width of compression flange, in.

$f_{pu}$  = Specified tensile strength of prestressing steel, ksi

$k$  =  $2 \left( 1.04 - \frac{f_{py}}{f_{pu}} \right) = 0.28$  for low-relaxation strand.

$d_p$  = Distance from extreme compression fiber to the centroid of the prestressing tendons, in.

$h_f$  = Depth of compression flange, in.

If the condition in Equation 4.106 is satisfied, the nominal flexural resistance of the rectangular section is given as:

$$M_n = A_{ps} f_{ps} \left( d_p - \frac{a}{2} \right) \quad (4.107)$$

where:

$M_n$  = Nominal flexural moment resistance, k-ft.

$f_{ps}$  = Average stress in prestressing steel (see Equation 4.88), ksi

$d_p$  = Distance from extreme compression fiber to the centroid of the prestressing tendons, in.

$a$  =  $\beta_1 c$

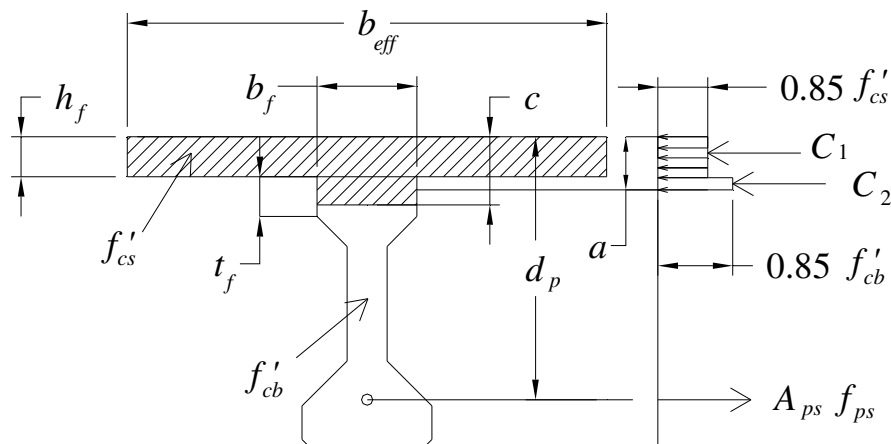


The Equations 4.108 and 4.109 are based on the following assumptions.

1. The section is transformed and the concrete strengths of the transformed slab and the girder are equal. (This assumption is not considered for the parametric study to establish a more accurate estimate of the design flexural moment resistance.)
2. The thickness of the web is constant. (This assumption is also not valid for the parametric study for I-shaped sections because the neutral axis might fall in the flange of the girder, the fillet portion, or the web.)

Considering the above stated reasons, formulas for determining the depth of the neutral axis and the design flexural strength of the section are developed for different cases in the parametric study, as outlined below.

Case I considers the neutral axis lies in the flange of the girder, as shown in Figure 4.12.



**Figure 4.12. Neutral Axis lies in the Girder Flange**

Using the AASHTO LRFD approach to multiply the flange compression force with the stress block factor,  $\beta_1$ , gives the following expressions.

$$C_1 = 0.85 f'_{cs} \beta_{1avg} b_{eff} h_f \quad (4.110)$$

$$C_2 = 0.85 f'_{cb} b_f \beta_{1avg} (c - h_f) \quad (4.111)$$

$$T = A_{ps} f_{ps} \quad (4.112)$$

From Equation 4.88,  $f_{ps} = f_{pu} \left( 1 - k \frac{c}{d_p} \right)$

$$T = A_{ps} f_{pu} \left( 1 - k \frac{c}{d_p} \right) \quad (4.113)$$

Applying equilibrium and solving for the neutral axis depth gives the following.

$$T = C_1 + C_2 \quad (4.114)$$

$$c = \frac{A_{ps} f_{pu} - 0.85 h_f \beta_{1avg} (f'_{cs} b_{eff} - f'_{cb} b_f)}{0.85 f'_{cb} \beta_{1avg} b_f + k A_{ps} \frac{f_{pu}}{d_p}} \leq h_f + t_f \quad (4.115)$$

Taking moments about  $C_1$ , the reduced nominal flexural moment strength at the section is as follows.

$$\phi M_n = \phi [T(d_p - 0.5h_f) - C_2(0.5a)] \quad (4.116)$$

where:

$T$  = Tensile force in the prestressing strands, kips

$C_1$  = Compression force in the slab, kips

$C_2$  = Compression force in the girder flange within stress block depth, kips

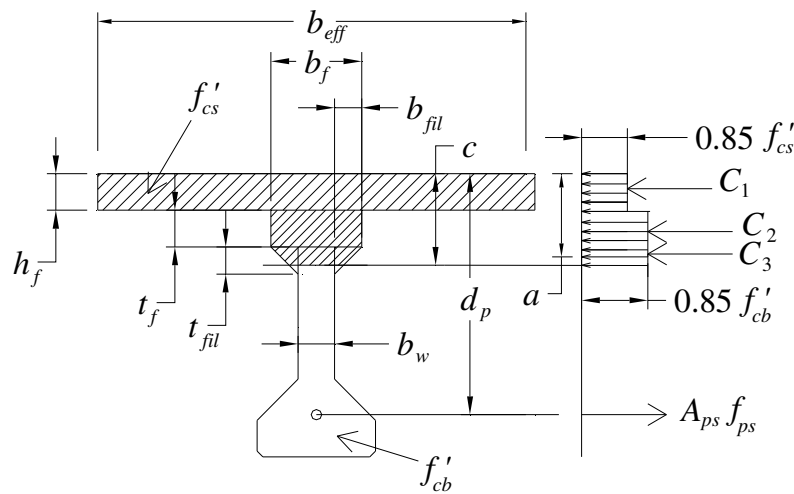
$f'_{cs}$  = Flange (slab) concrete strength at service, ksi

$f'_{cb}$  = Girder concrete strength at service, ksi

$b_{eff}$  = Effective flange width, in.

- $b_f$  = Girder flange width, in.  
 $h_f$  = Thickness of slab, in.  
 $\beta_{1avg}$  = Stress block factor (see Equation 4.86)

Case II considers the neutral axis lies in the fillet portion of the girder as shown in Figure 4.13. This case was ignored for the Standard Specifications as the fillet portion does not affect the design moment resistance significantly. However, it was found that ignoring the fillet contribution changes the depth of the neutral axis significantly. As the ductility limits for the LRFD Specifications are based on the depth of the neutral axis, the fillet contribution for estimating the neutral axis depth, cannot be ignored. However, for moment calculations, the fillet contribution is ignored for simplicity and because it has little effect on the nominal moment capacity.



**Figure 4.13. Neutral Axis lies in the Fillet Portion of the Girder**

Using the LRFD approach to multiply the flange compression with the factor  $\beta_{1avg}$  gives the following expressions.

$$C_1 = 0.85 f'_{cs} \beta_{1avg} b_{eff} h_f \quad (4.117)$$

$$C_2 = 0.85 f'_{cb} \beta_{1avg} b_f t_f \quad (4.118)$$

when  $c \leq h_f + t_f + t_{fil}$ :

$$C_3 = 0.85 f'_{cb} \beta_{1avg} (c - h_f - t_f) \left\{ b_w + b_{fil} + b_{fil} \frac{[t_{fil} - \beta_{1avg} (c - h_f - t_f)]}{t_{fil}} \right\} \quad (4.119)$$

$$T = A_{ps} f_{ps} \quad (4.120)$$

From equation 4.88,  $f_{ps} = f_{pu} \left( 1 - k \frac{c}{d_p} \right)$

$$T = A_{ps} f_{pu} \left( 1 - k \frac{c}{d_p} \right) \quad (4.121)$$

Applying the equilibrium gives the following expression.

$$T = C_1 + C_2 + C_3 \quad (4.122)$$

Imposing the equilibrium condition in Equation 4.122 results in a quadratic equation in  $c$ , which is solved using trial and error method. The program for the parametric study first assumes the value of  $c$  as  $c = h_f + t_f$  and checks the equilibrium equation, if the equilibrium is not satisfied the depth of neutral axis is incremented by 0.1 in. and the equilibrium is checked. This process is continued until the depth of neutral axis  $c$ , corresponding to the equilibrium condition, is established.

For the moment resistance calculation, the fillet contribution is neglected. The Equation 4.119 is modified to the following.

$$C_3 = 0.85 f'_{cb} \beta_{1avg} (c - h_f - t_f) b_w \quad (4.123)$$

Taking moments about  $C_1$ , the design flexural strength at the section can be given as:

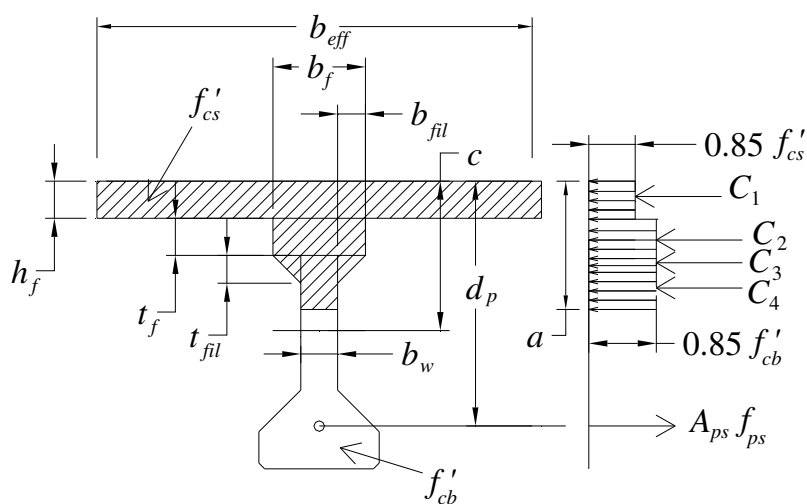
$$\phi M_n = \phi [T (d_p - 0.5h_f) - C_2 (0.5h_f + 0.5t_f) - C_3 (0.5a + 0.5t_f)] \quad (4.124)$$



where:

- $T$  = Tensile force in the prestressing strands, kips  
 $C_1$  = Compression force in the slab, kips  
 $C_2$  = Compression force in the girder flange, kips  
 $C_3$  = Compression force in the girder web within the stress block depth, kips (see Equation 4.123)  
 $f'_{cs}$  = Flange (slab) concrete strength at service, ksi  
 $f'_{cb}$  = Girder concrete strength at service, ksi  
 $b_{eff}$  = Effective flange width, in.  
 $b_f$  = Flange width of the girder, in.  
 $h_f$  = Thickness of the slab, in.  
 $t_{fil}$  = Thickness of the girder fillet, in.  
 $b_{fil}$  = Girder fillet width, in.  
 $\beta_{1avg}$  = Stress block factor (see Equation 4.86)

Case III considers the neutral axis lies in the web portion of the girder as shown in Figure 4.14.



**Figure 4.14. Neutral Axis lies in the Web Portion of the Girder**

Using the LRFD Specifications approach to multiply the flange compression with the factor  $\beta_{1avg}$  gives the following expressions.

$$C_1 = 0.85 f'_{cs} \beta_{1avg} b_{eff} h_f \quad (4.125)$$

$$C_2 = 0.85 f'_{cb} \beta_{1avg} b_f t_f \quad (4.126)$$

$$C_3 = 0.85 f'_{cb} \beta_{1avg} t_{fil} (b_w + b_{fil}) \quad (4.127)$$

$$C_4 = 0.85 f'_{cb} \beta_{1avg} (c - h_f - t_f - t_{fil}) b_w \quad (4.128)$$

$$T = A_{ps} f_{ps} \quad (4.129)$$

From Equation 4.88,  $f_{ps} = f_{pu} \left( 1 - k \frac{c}{d_p} \right)$

$$T = A_{ps} f_{pu} \left( 1 - k \frac{c}{d_p} \right) \quad (4.130)$$

Applying the equilibrium condition gives the following expression.

$$T = C_1 + C_2 + C_3 + C_4 \quad (4.131)$$

$$c = \frac{A_{ps} f_{pu} - 0.85 h_f \beta_{1avg} (f'_{cs} b_{eff} - f'_{cb} b_w) - 0.85 f'_{cb} \beta_{1avg} t_f (b_f - b_w) - 0.85 f'_{cb} \beta_{1avg} t_{fil} b_{fil}}{0.85 f'_{cb} \beta_{1avg} b_w + k A_{ps} \frac{f_{pu}}{d_p}} \quad \dots(4.132)$$

Taking moments about  $C_1$ , the reduced nominal flexural strength at the section can be given as

$$\phi M_n = \phi [T (d_p - 0.5 h_f) - C_2 (0.5 h_f + 0.5 t_f) - C_3 \left\{ \frac{t_{fil}}{3} \left[ \frac{3 b_w + 2 b_{fil}}{2 b_w + 2 b_{fil}} \right] + t_f + 0.5 h_f \right\} - C_4 (0.5 a + 0.5 t_f + 0.5 t_{fil})] \quad (4.133)$$

where:

- $C_3$  = Compression force in the fillet and the web between fillets, kips
- $C_4$  = Compression force in the web portion (excluding web portion between the fillets) under the stress block, kips
- $t_{fil}$  = Thickness of girder fillet, in.
- $b_{fil}$  = Girder fillet width, in.
- $b_w$  = Girder web width, in.
- $\beta_{1avg}$  = Stress block factor (see Equation 4.86)

#### 4.4.6.6 Maximum Steel Reinforcement

The AASHTO Standard Specifications requires that the maximum prestressing steel be limited to ensure yielding of steel when ultimate capacity is reached. The Standard Specifications Article 9.18.1 specifies the limits of reinforcement as follows.

Reinforcement index for rectangular sections:

$$\frac{\rho^* f_{su}^*}{f_c'} < 0.36\beta_1 \quad (4.134)$$

Reinforcement index for flanged sections:

$$\frac{A_{sr} f_{su}^*}{b' d f_c'} < 0.36\beta_1 \quad (4.135)$$

If the above maximum reinforcement limits are not satisfied, the Standard Specifications recommend the design flexural moment strength of the girder to be limited as follows.

For rectangular sections:

$$\phi M_n = \phi [(0.36\beta_1 - 0.08\beta_1^2) f_c' b d^2] \quad (4.136)$$

For flanged sections:

$$\phi M_n = \phi [(0.36\beta_1 - 0.08\beta_1^2) f'_c b d^2 + 0.85f'_c(b - b') t (d - 0.5t)] \quad (4.137)$$

where:

$\rho^*$  = Ratio of prestressing reinforcement

$$= \frac{A_s^*}{bd}$$

$A_s^*$  = Area of pretensioned reinforcement, in.<sup>2</sup>

$b$  = Effective flange width, in.

$d$  = Distance from extreme compression fiber to the centroid of the prestressing force, in.

$f_{su}^*$  = Average stress in prestressing steel at ultimate load, ksi

$f'_c$  = Compressive strength of slab concrete at service, ksi

$\beta_1$  = Stress block factor

$A_{sr}$  = Steel area required to develop the compressive strength of the overhanging portions of the slab, in.<sup>2</sup>

$\phi$  = Resistance factor specified as 1.0 for flexure of prestressed concrete member

$M_n$  = Nominal moment strength at the section, k-ft.

$b'$  = Width of web of a flanged member, in.

$t$  = Average thickness of the flange, in.

The above flexural moment strength limit provided by the Standard Specifications for flanged section behavior is based on the transformed section. However for the parametric study a transformed section was not considered because refined equations were developed for computation of moment strength of the flanged sections. Hence, a conservative estimate of the design flexural strength can be made by using the concrete strength of the slab for the entire flanged section as the concrete strength of slab is less than the girder concrete strength. This method is used in the parametric study.

LRFD Specifications Article 5.7.3.3.1 specifies the maximum total amount of prestressed and non-prestressed reinforcement to be limited such that

$$\frac{c}{d_e} \leq 0.42 \quad (4.138)$$

where:

$$\begin{aligned} c &= \text{Distance from the extreme compression fiber to the neutral axis, in.} \\ d_e &= \text{Corresponding effective depth from the extreme compression fiber to the} \\ &\quad \text{centroid of the tensile force in the tensile reinforcement, in.} \\ &= \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y} \end{aligned} \quad (4.139)$$

The parametric study only considers fully prestressed sections, for which the effective depth  $d_e$  reduces to  $d_p$ . In case the above limit is not satisfied, the following equations are provided in the LRFD Specifications to limit the flexural resistance of the girder section.

For rectangular sections:

$$M_n = [(0.36\beta_{1avg} - 0.08\beta_{1avg}^2) f'_c b d_e^2] \quad (4.140)$$

For flanged sections:

$$M_n = [(0.36\beta_{1avg} - 0.08\beta_{1avg}^2) f'_c b_w d_e^2 + 0.85\beta_{1avg} f'_c (b-b_w) h_f (d_e - 0.5h_f)] \quad (4.141)$$

where:

$$\begin{aligned} M_n &= \text{Nominal moment strength at the section, k-ft.} \\ b &= \text{Effective flange width, in.} \\ f'_c &= \text{Compressive strength of slab concrete at service, ksi} \\ \beta_{1avg} &= \text{Stress block factor (see Equation 4.86)} \end{aligned}$$

$b_w$  = Width of web of a flanged member, in.

$h_f$  = Compression flange depth, in.

Additional variables are the same as defined for Equation 4.137.

The flexural moment strength limit provided by LRFD Specifications for flanged section behavior is based on the transformed section. However for the parametric study, transformed section was not considered. Hence, a conservative estimate of the design flexural strength can be made by using the concrete strength of the slab. This method is used in the parametric study.

#### 4.4.6.7 Minimum Steel Reinforcement

The Standard Specifications Article 9.18.2 requires the minimum amount of prestressed and non-prestressed reinforcement to be adequate to develop an ultimate moment at the critical section of at least 1.2 times the cracking moment,  $M_{cr}^*$ .

$$\phi M_n \geq 1.2 M_{cr}^* \quad (4.142)$$

where:

$M_n$  = Nominal flexural moment strength, k-in.

$M_{cr}^*$  = Cracking moment, k-in.

$$= (f_r + f_{pe}) S_c - M_{dnc} \left( \frac{S_c}{S_b} - 1 \right) \quad (4.143)$$

$f_r$  = Modulus of rupture, psi

$$= 7.5 \sqrt{f'_c} \text{ for normal weight concrete}$$

$f'_c$  = Compressive strength of girder concrete at service, psi

$f_{pe}$  = Compressive stress in concrete due to effective prestress force at extreme fiber of section where tensile stress is caused by externally applied loads, ksi

$$= \frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b}$$

$P_{se}$  = Effective prestress force after losses, kips

$A$  = Area of cross-section, in.<sup>2</sup>

$e_c$  = Eccentricity of prestressing strands at midspan, in.

$S_b$  = Section modulus of non-composite section referenced to the extreme fiber where tensile stress is caused by externally applied loads, in.<sup>3</sup>

$S_c$  = Section modulus of composite section referenced to the extreme fiber where tensile stress is caused by externally applied loads, in.<sup>3</sup>

$M_{d/nc}$  = Non-composite dead load moment at midspan due to self weight of girder and weight of slab, k-in.

The above limit is waived at the sections where the area of prestressed and non-prestressed reinforcement provided is at least one-third greater than that required by analysis based on the loading combinations.

The LRFD Specifications Article 5.7.3.3.2 specifies the minimum amount of prestressed and non-prestressed tensile reinforcement such that a factored flexural resistance,  $M_r$  is at least equal to

- 1.2 times the cracking moment,  $M_{cr}$ , determined on the basis of elastic stress distribution and the modulus of rupture  $f_r$  of the concrete, and
- 1.33 times the factored moment required by the applicable strength load combination.

The cracking moment is given by the following formula.

$$M_{cr} = (f_r + f_{cpe}) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \geq S_c f_r \quad (4.144)$$

The LRFD Specifications has a typographical error ( $\geq$  mistyped as  $\leq$ ) in the above equation.

where:

$M_{cr}$  = Cracking moment, k-in.

$f_{cpe}$  = Compressive stress in concrete due to effective prestress forces at extreme fiber of section where tensile stress is caused by externally applied loads, ksi

$$= \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b}$$

$P_{pe}$  = Effective prestress force after losses, kips

$e_c$  = Eccentricity of prestressing strands at midspan, in.

$M_{dnc}$  = Total unfactored non-composite dead load moment, k-in.

$S_{nc}$  = Section modulus referenced to the extreme fiber of the non-composite section where tensile stress is caused by externally applied loads, in.<sup>3</sup>

$S_c$  = Section modulus referenced to the extreme fiber of the composite section where tensile stress is caused by externally applied loads, in.<sup>3</sup>

$f_r$  = Rupture modulus specified as  $0.24\sqrt{f'_c}$  for normal-weight concrete, ksi

$f'_c$  = Compressive strength of girder concrete at service, ksi

#### 4.4.7 Shear Design

Shear design of a composite prestressed concrete girder consists of the design for both transverse and interface shear, as outlined in the following sections.

##### 4.4.7.1 Transverse Shear Design

The AASHTO Standard and LRFD Specifications require that prestressed concrete flexural members be reinforced for shear and diagonal tension stresses. However, the two specifications follow different methodologies to predict the shear strength of member. The Standard Specifications uses the constant 45° truss analogy to predict the shear behavior of the member where concrete in compression acts as struts and tension steel acts as ties. The LRFD Specifications use a variable angle truss analogy



with modified compression strength of concrete popularly known as “Modified Compression Field Theory (MCFT).” The two theories are discussed in detail in Section 7. The following sections outline the procedures for shear design of prestressed concrete girders specified by the AASHTO Standard and LRFD Specifications.

**4.4.7.1.1 Standard Specifications.** The following section outlines the shear design procedures specified by the Standard Specifications Article 9.20. Shear reinforcement is not necessary if the following condition is met.

$$V_u < \frac{\phi V_c}{2} \quad (4.145)$$

where:

- $V_u$  = Factored shear force at the section, kips
- =  $1.3(V_d + 1.67 V_{LL+I})$  for this study
- $V_d$  = Shear force at the section due to dead loads, kips
- $V_{LL+I}$  = Shear force at the section due to live load including impact load, kips
- $V_c$  = Nominal shear strength provided by the concrete, taken as lesser of  $V_{ci}$  (see Equation 4.147) and  $V_{cw}$  (see Equation 4.149), kips
- $\phi$  = Strength reduction factor, specified as 0.9 for shear of prestressed concrete members

If the condition in Equation 4.145 is not satisfied, the member shall be designed such that:

$$V_u \leq \phi (V_c + V_s) \quad (4.146)$$

where:

- $V_s$  = Nominal shear strength provided by the web reinforcement, kips

The shear design in the parametric study is carried out at the critical section for shear. The critical section is specified as  $h_c/2$  from the face of the support, where  $h_c$  is the depth of the composite section. However, as the support dimensions are unknown in this study the critical section is calculated from the center line of the bearing support which yields a slightly conservative estimate of the required web reinforcement. The shear strength provided by normal weight concrete is calculated using the following expressions.

$$V_{ci} = 0.6\sqrt{f'_c} b' d + V_d + \frac{V_i M_{cr}}{M_{max}} \geq 1.7 \sqrt{f'_c} b' d \quad (4.147)$$

$$V_{cw} = (3.5 \sqrt{f'_c} + 0.3 f_{pc}) b' d + V_p \quad (4.148)$$

where:

- $V_{ci}$  = Nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, kips
- $V_{cw}$  = Nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in the web, kips
- $f'_c$  = Compressive strength of girder concrete at service, ksi
- $b'$  = Width of the web of a flanged member, in.
- $d$  = Distance from extreme compressive fiber to centroid of pretensioned reinforcement, but not less than  $0.8h_c$ , in.
- $h_c$  = Depth of composite section, in.
- $V_d$  = Shear force at the section due to dead loads, kips
- $V_i$  = Factored shear force at the section due to externally applied loads occurring simultaneously with  $M_{max}$ , kips  
 $= V_{mu} - V_d$
- $V_{mu}$  = Factored shear force occurring simultaneously with  $M_u$ , conservatively taken as maximum shear force at the section, kips

$M_{cr}$  = Moment causing flexural cracking of section due to externally applied loads, k-in.

$$= (6\sqrt{f'_c} + f_{pe} - f_d) \frac{I}{Y_t}$$

$f_{pe}$  = Compressive stress in concrete due to effective pretension force at extreme fiber of section where tensile stress is caused by externally applied loads i.e. bottom fiber of the girder in present case, ksi

$$= \frac{P_{se}}{A} + \frac{P_{se}e_x}{S_b}$$

$f_{pc}$  = Compressive stress in concrete at centroid of the cross-section resisting externally applied loads, ksi

$$= \frac{P_{se}}{A} - \frac{P_{se}e_x(y_{bcomp} - y_b)}{I} + \frac{M_D(y_{bcomp} - y_b)}{I}$$

$P_{se}$  = Effective prestress force after all losses. If the section at a distance  $h_c/2$  from the face of the support is closer to the girder end than the transfer length (50 strand diameters) the prestressing force is assumed to vary linearly from 0 at the end to maximum at a the transfer length section, kips

$e_x$  = Eccentricity of the strands at the section considered, in.

$A$  = Area of girder cross-section, in.<sup>2</sup>

$S_b$  = Section modulus referenced to the extreme bottom fiber of the non-composite section, in.<sup>3</sup>

$f_d$  = Stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, ksi

$$= \left[ \frac{M_g + M_s}{S_b} + \frac{M_{SDL}}{S_{bc}} \right]$$

$M_g$  = Unfactored bending moment due to girder self-weight, k-in.

$M_s$  = Unfactored bending moment due to slab weight, k-in.

$M_{SDL}$  = Unfactored superimposed dead load moment, k-in.

- $S_{bc}$  = Composite section modulus referenced to the extreme bottom fiber of the precast girder, in.<sup>3</sup>  
 $M_{max}$  = Maximum factored moment at the section due to externally applied loads, k-in.  
 $\quad = M_u - M_d$   
 $M_u$  = Factored bending moment at the section, k-in.  
 $\quad = 1.3(M_d + 1.67 M_{LL+I})$   
 $M_d$  = Bending moment at section due to unfactored dead loads, k-in.  
 $M_{LL+I}$  = Bending moment at section due to live load including impact load, k-in.  
 $I$  = Moment of inertia of the girder cross-section, in.<sup>4</sup>  
 $y_{bcomp}$  = Lesser of  $y_{bc}$  and the distance from bottom fiber of the girder to the junction of the web and top flange, in.  
 $y_{bc}$  = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder, in.  
 $y_b$  = Distance from centroid to the extreme bottom fiber of the non-composite precast girder, in.  
 $M_D$  = Moment due to unfactored non-composite dead loads at the critical section, k-in.  
 $V_p$  = Vertical component of prestress force for harped strands, kips  
 $\quad = P_{se} \sin \Psi$   
 $\Psi$  = Angle of the harped tendons to the horizontal, radians

The area of the web reinforcement shall be provided such that the condition in Equation 4.146 is satisfied. The nominal shear strength provided by steel reinforcement,  $V_s$ , is calculated using the following expression.

$$V_s = \frac{A_v f_y d}{s} < 8 \sqrt{f'_c} b' d \quad (4.149)$$

The minimum are of web reinforcement is limited to the following value.

$$A_{v \min} = \frac{50b's}{f_y} \quad (4.150)$$

where:

- $A_v$  = Area of web reinforcement, in.<sup>2</sup>
- $b'$  = Width of web of a flanged member, in.
- $f'_c$  = Compressive strength of girder concrete at service, ksi
- $s$  = Spacing of web reinforcement, in.
- $f_y$  = Yield strength of web reinforcement, ksi
- $d$  = Distance from extreme compressive fiber to centroid of pretensioned reinforcement, but not less than  $0.8h_c$ , in.
- $h_c$  = depth of composite section, in.

The spacing of the web reinforcement shall not exceed  $0.75h_c$  or 24 in. If  $V_s$  exceeds  $4\sqrt{f'_c} b' d$  the maximum spacing shall be reduced by one-half.

**4.4.7.1.2 LRFD Specifications.** The LRFD Specifications uses the Modified Compression Field Theory (MCFT) for the shear design provisions. The MCFT takes into account different factors such as strain condition of the section, and shear stress in the concrete to predict the shear strength of the section. This theory is believed to yield a more realistic estimate of the shear strength of the concrete (see Sec. 7.2.2). The shear strength of concrete is approximated based on a parameter  $\beta$ . The critical section for shear is calculated based on the angle of inclination of the diagonal compressive stress,  $\theta$ . If the values of these parameters are taken as  $\theta = 45^\circ$  and  $\beta = 2$ , the theory will yield the results similar to the  $45^\circ$  truss analogy method employed in the Standard Specifications. The provisions for transverse shear design in the LRFD Specifications are outlined in this section. The LRFD Specifications specifies that transverse reinforcement is needed at sections with the following condition.

$$V_u > 0.5 \phi (V_c + V_p) \quad (4.151)$$

where:

- $V_u$  = Factored shear force at the section, kips  
 $= 1.25(DC) + 1.5(DW) + 1.75(LL + IM)$  for this study
- $DC$  = Shear force at the section due to dead loads except wearing surface weight, kips
- $DW$  = Shear force at the section due to wearing surface weight, kips
- $LL+IM$  = Shear force at the section due to live load including impact, kips
- $V_c$  = Nominal shear strength provided by concrete, kips
- $\phi$  = Strength reduction factor specified as 0.9 for shear of prestressed concrete members.
- $V_p$  = Component of prestressing force in the direction of shear force, kips

The AASHTO LRFD Specifications specifies the critical section for shear near the supports as the larger value of  $0.5d_v \cot \theta$  or  $d_v$ , measured from the face of the support.

where:

- $d_v$  = Effective shear depth, in.  
 $=$  Distance between resultants of tensile and compressive forces,  $(d_e - a/2)$ , but not less than the greater of  $(0.9d_e)$  or  $(0.72h)$ , in.
- $d_e$  = Corresponding effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement, in.
- $a$  = Depth of compression block, in.
- $h$  = Depth of composite section, in.
- $\theta$  = Angle of inclination of diagonal compressive stresses (slope of compression field). The value of  $\theta$  is unknown and is assumed to be  $23^\circ$  at the beginning of design, and iterations are made until it converges to a particular value.

The nominal shear resistance at a section is lesser of the following two values

$$V_n = (V_c + V_s + V_p) \text{ and} \quad (4.152)$$

$$V_n = 0.25 f'_c b_v d_v + V_p \quad (4.153)$$

Shear resistance provided by the concrete,  $V_c$ , is given as

$$V_c = 0.0316 \beta \sqrt{f'_c} b_v d_v \quad (4.154)$$

Shear resistance provided by transverse steel reinforcement,  $V_s$ , is given as

$$V_s = \frac{A_v f_y d_v (\cot \theta + \cot \alpha) \sin \alpha}{s} \quad (4.155)$$

where:

- $d_v$  = Effective shear depth, in.
- $b_v$  = Girder web width, in
- $f'_c$  = Girder concrete strength at service, ksi
- $V_p$  = Component of prestressing force in the direction of shear force, kips
- $\beta$  = Factor indicating ability of diagonally cracked concrete to transfer tension.
- $\theta$  = Angle of inclination of diagonal compressive stresses (slope of compression field), radians
- $A_v$  = Area of shear reinforcement within a distance  $s$ , in.<sup>2</sup>
- $s$  = Spacing of stirrups, in.
- $f_y$  = Yield strength of shear reinforcement, ksi
- $\alpha$  = Angle of inclination of transverse reinforcement to longitudinal axis, taken as 90° for vertical stirrups

### Determination of $\beta$ and $\theta$

The values of  $\beta$  and  $\theta$  depend on the shear stress in the concrete,  $v_u$  and the longitudinal strain,  $\epsilon_x$  of the section. The shear stress in the concrete is given as

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} \quad (4.156)$$

where:

- $v_u$  = Shear stress in concrete, ksi
- $V_u$  = Factored shear force at the section, kips
- $\phi$  = Resistance factor, specified as 0.9 for prestressed concrete members
- $V_p$  = Component of prestressing force in the direction of shear force, kips
- $b_v$  = Girder web width, in.
- $d_v$  = Effective shear depth, in.

For the sections containing at least the minimum transverse reinforcement the longitudinal strain,  $\epsilon_x$  is determined as follows.

$$\epsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_p A_{ps})} \leq 0.001 \quad (4.157)$$

For the sections containing less than minimum transverse reinforcement the longitudinal strain,  $\epsilon_x$  is found using the following expression.

$$\epsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{E_p A_{ps}} \leq 0.002 \quad (4.158)$$

If the value of  $\epsilon_x$  is negative, the longitudinal strain is

$$\epsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_c A_c + E_p A_{ps})} \quad (4.159)$$



where:

- $V_u$  = Applied factored shear force at the specified section, kips
- $M_u$  = Applied factored moment at the specified section  $> V_u d_v$ , k-in.
- $N_u$  = Applied factored normal force at the specified section, kips
- $A_c$  = Area of the concrete on the flexural tension side of the member, in.<sup>2</sup>
- $A_{ps}$  = Area of prestressing steel on the flexural side of the member, in.<sup>2</sup>
- $f_{po}$  = Parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi). LRFD Article C5.8.3.4.2 recommends that for pretensioned members,  $f_{po}$  be taken as the stress in strands when the concrete is cast around them, which is approximately  $0.7f_{pu}$ , ksi
- $f_{pu}$  = Ultimate strength of prestressing strands, ksi
- $V_p$  = Vertical component of prestress force for harped strands, kips

For the sections containing less than minimum transverse reinforcement, the crack spacing parameter  $s_{xe}$  is required to determine the parameters  $\beta$  and  $\theta$ . The crack spacing parameter  $s_{xe}$  shall be calculated as follows

$$s_{xe} = s_x \frac{1.38}{a_g + 0.63} \leq 80 \text{ in.} \quad (4.160)$$

where:

- $a_g$  = Maximum aggregate size, in.
- $s_x$  = Lesser of either  $d_v$  or the maximum distance between layers of longitudinal crack control reinforcement, in.

The parameters  $\beta$  and  $\theta$  are calculated by interpolating for the determined values of  $v_u$  and  $\epsilon_x$  from the table shown in Figure 4.15 taken from the LRFD Specifications (AASHTO 2004).

Table 5.8.3.4.2-1 Values of  $\theta$  and  $\beta$  for Sections with Transverse Reinforcement.

| $\frac{v_u}{f'_c}$ | $\epsilon_x \times 1,000$ |              |              |              |              |              |              |              |              |
|--------------------|---------------------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|
|                    | $\leq -0.20$              | $\leq -0.10$ | $\leq -0.05$ | $\leq 0$     | $\leq 0.125$ | $\leq 0.25$  | $\leq 0.50$  | $\leq 0.75$  | $\leq 1.00$  |
| $\leq 0.075$       | 22.3<br>6.32              | 20.4<br>4.75 | 21.0<br>4.10 | 21.8<br>3.75 | 24.3<br>3.24 | 26.6<br>2.94 | 30.5<br>2.59 | 33.7<br>2.38 | 36.4<br>2.23 |
| $\leq 0.100$       | 18.1<br>3.79              | 20.4<br>3.38 | 21.4<br>3.24 | 22.5<br>3.14 | 24.9<br>2.91 | 27.1<br>2.75 | 30.8<br>2.50 | 34.0<br>2.32 | 36.7<br>2.18 |
| $\leq 0.125$       | 19.9<br>3.18              | 21.9<br>2.99 | 22.8<br>2.94 | 23.7<br>2.87 | 25.9<br>2.74 | 27.9<br>2.62 | 31.4<br>2.42 | 34.4<br>2.26 | 37.0<br>2.13 |
| $\leq 0.150$       | 21.6<br>2.88              | 23.3<br>2.79 | 24.2<br>2.78 | 25.0<br>2.72 | 26.9<br>2.60 | 28.8<br>2.52 | 32.1<br>2.36 | 34.9<br>2.21 | 37.3<br>2.08 |
| $\leq 0.175$       | 23.2<br>2.73              | 24.7<br>2.66 | 25.5<br>2.65 | 26.2<br>2.60 | 28.0<br>2.52 | 29.7<br>2.44 | 32.7<br>2.28 | 35.2<br>2.14 | 36.8<br>1.96 |
| $\leq 0.200$       | 24.7<br>2.63              | 26.1<br>2.59 | 26.7<br>2.52 | 27.4<br>2.51 | 29.0<br>2.43 | 30.6<br>2.37 | 32.8<br>2.14 | 34.5<br>1.94 | 36.1<br>1.79 |
| $\leq 0.225$       | 26.1<br>2.53              | 27.3<br>2.45 | 27.9<br>2.42 | 28.5<br>2.40 | 30.0<br>2.34 | 30.8<br>2.14 | 32.3<br>1.86 | 34.0<br>1.73 | 35.7<br>1.64 |
| $\leq 0.250$       | 27.5<br>2.39              | 28.6<br>2.39 | 29.1<br>2.33 | 29.7<br>2.33 | 30.6<br>2.12 | 31.3<br>1.93 | 32.8<br>1.70 | 34.3<br>1.58 | 35.8<br>1.50 |

Table 5.8.3.4.2-2 Values of  $\theta$  and  $\beta$  for Sections with Less than Minimum Transverse Reinforcement.

| $s_{xe}$<br>(in.) | $\epsilon_x \times 1000$ |              |              |              |              |              |              |              |              |              |              |
|-------------------|--------------------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|
|                   | $\leq -0.20$             | $\leq -0.10$ | $\leq -0.05$ | $\leq 0$     | $\leq 0.125$ | $\leq 0.25$  | $\leq 0.50$  | $\leq 0.75$  | $\leq 1.00$  | $\leq 1.50$  | $\leq 2.00$  |
| $\leq 5$          | 25.4<br>6.36             | 25.5<br>6.06 | 25.9<br>5.56 | 26.4<br>5.15 | 27.7<br>4.41 | 28.9<br>3.91 | 30.9<br>3.26 | 32.4<br>2.86 | 33.7<br>2.58 | 35.6<br>2.21 | 37.2<br>1.96 |
| $\leq 10$         | 27.6<br>5.78             | 27.6<br>5.78 | 28.3<br>5.38 | 29.3<br>4.89 | 31.6<br>4.05 | 33.5<br>3.52 | 36.3<br>2.88 | 38.4<br>2.50 | 40.1<br>2.23 | 42.7<br>1.88 | 44.7<br>1.65 |
| $\leq 15$         | 29.5<br>5.34             | 29.5<br>5.34 | 29.7<br>5.27 | 31.1<br>4.73 | 34.1<br>3.82 | 36.5<br>3.28 | 39.9<br>2.64 | 42.4<br>2.26 | 44.4<br>2.01 | 47.4<br>1.68 | 49.7<br>1.46 |
| $\leq 20$         | 31.2<br>4.99             | 31.2<br>4.99 | 31.2<br>4.99 | 32.3<br>4.61 | 36.0<br>3.65 | 38.8<br>3.09 | 42.7<br>2.46 | 45.5<br>2.09 | 47.6<br>1.85 | 50.9<br>1.52 | 53.4<br>1.31 |
| $\leq 30$         | 34.1<br>4.46             | 34.1<br>4.46 | 34.1<br>4.46 | 34.2<br>4.43 | 38.9<br>3.39 | 42.3<br>2.82 | 46.9<br>2.19 | 50.1<br>1.84 | 52.6<br>1.60 | 56.3<br>1.30 | 59.0<br>1.10 |
| $\leq 40$         | 36.6<br>4.06             | 36.6<br>4.06 | 36.6<br>4.06 | 36.6<br>4.06 | 41.2<br>3.20 | 45.0<br>2.62 | 50.2<br>2.00 | 53.7<br>1.66 | 56.3<br>1.43 | 60.2<br>1.14 | 63.0<br>0.95 |
| $\leq 60$         | 40.8<br>3.50             | 40.8<br>3.50 | 40.8<br>3.50 | 40.8<br>3.50 | 44.5<br>2.92 | 49.2<br>2.32 | 55.1<br>1.72 | 58.9<br>1.40 | 61.8<br>1.18 | 65.8<br>0.92 | 68.6<br>0.75 |
| $\leq 80$         | 44.3<br>3.10             | 44.3<br>3.10 | 44.3<br>3.10 | 44.3<br>3.10 | 47.1<br>2.71 | 52.3<br>2.11 | 58.7<br>1.52 | 62.8<br>1.21 | 65.7<br>1.01 | 69.7<br>0.76 | 72.4<br>0.62 |

Figure 4.15. Values of  $\beta$  and  $\theta$  - AASHTO LRFD Specifications (AASHTO 2004)

The maximum spacing,  $s$ , of transverse reinforcement is limited by the LRFD Specifications as follows:

if  $v_u < 0.125 f'_c$ :

$$s \leq 0.8d_v \leq 24.0 \text{ in.} \quad (4.161)$$

or if  $v_u \geq 0.125 f'_c$ :

$$s \leq 0.4d_v \leq 12.0 \text{ in.} \quad (4.162)$$

where:

- $s$  = Center-to-center spacing of shear reinforcement, in.
- $v_u$  = Shear stress in the concrete, ksi
- $d_v$  = Effective shear depth, in.

The minimum area of transverse reinforcement is given as

$$A_v \geq 0.0316 \sqrt{f'_c} \frac{b_v s}{f_y} \quad (4.163)$$

where:

- $A_v$  = Area of transverse shear reinforcement within spacing  $s$ , in.<sup>2</sup>
- $f'_c$  = Girder concrete strength at service, ksi
- $b_v$  = Girder web width, in.
- $f_y$  = Yield strength of shear reinforcement, ksi
- $s$  = Center-to-center spacing of shear reinforcement, in.

The LRFD Specifications requires that at each section the tensile capacity of the longitudinal reinforcement on the flexural tension side of the member must satisfy the following expression

$$A_s f_y + A_{ps} f_{ps} \geq \frac{M_u}{d_v \phi_f} + 0.5 \frac{N_u}{\phi_c} + \left( \frac{V_u}{\phi_v} - 0.5 V_s - V_p \right) \cot \theta \quad (4.164)$$

where:

- $A_s$  = Area of non-prestressed reinforcement on the flexural side of the member, in.<sup>2</sup>
- $A_{ps}$  = Area of prestressing steel on the flexural side of the member, in.<sup>2</sup>
- $f_y$  = Yield strength of non-prestressed reinforcement, ksi
- $f_{ps}$  = Effective stress in the prestressing steel, ksi
- $M_u$  = Applied factored moment at the specified section  $> V_u d_v$ , k-in.
- $N_u$  = Applied factored normal force at the specified section, kips
- $V_u$  = Applied factored shear force at the specified section, kips
- $V_s$  = Nominal shear strength provided by the web reinforcement, kips
- $V_p$  = Component of prestressing force in the direction of shear force, kips
- $d_v$  = Effective shear depth, in.
- $\theta$  = Angle of inclination of diagonal compressive stresses (slope of compression field), radians
- $\phi_f$  = Resistance factor for flexure, specified as 1.0 for prestressed concrete members
- $\phi_c$  = Resistance factor for axial force, specified as 0.75 for compression and 1.0 for tension in prestressed concrete members
- $\phi_v$  = Resistance factor for shear, specified as 0.9 for prestressed concrete members

The condition in Equation 4.165 is checked at the critical section for shear in the parametric study.

#### 4.4.7.2 Interface Shear Design

AASHTO Standard Specifications Article 9.20.4 specifies the requirements for horizontal shear design. The Standard Specifications also allow the use of refined methods for the interface shear design that are in agreement with comprehensive test results. The provisions of Article 9.20.4, outlined in this section, are used for the parametric study.

The horizontal shear design must satisfy the following expression.

$$V_u \leq \phi V_{nh} \quad (4.165)$$

where:

$V_u$  = Factored shear force at the section, kips

$\phi$  = Resistance factor specified as 0.90 for shear in prestressed concrete members

$V_{nh}$  = Nominal horizontal shear strength at the section, kips

The critical section for horizontal shear is at a distance of  $h_c/2$  from the center line of the support where  $h_c$  is the depth of the composite section (in.). The nominal horizontal shear strength must be calculated based on one of the following cases.

Case (a): Contact surface is clean, free of laitance, and intentionally roughened.

$$V_{nh} = 80 b_v d \quad (4.166)$$

Case (b): Minimum ties are used, contact surface is clean, free of laitance, but not intentionally roughened.

$$V_{nh} = 80 b_v d \quad (4.167)$$

Case (c): Minimum ties are used, contact surface is clean, free of laitance, and intentionally roughened to a full amplitude of 1/4 in.

$$V_{nh} = 350 b_v d \quad (4.168)$$

Case (d): For each percent of tie reinforcement crossing the contact surface in excess of the minimum requirement,  $V_{nh}$  may be increased by

$$\frac{160 f_y}{40,000} b_v d \quad (4.169)$$

where:

- $b_v$  = Width of cross-section at the contact surface being investigated for horizontal shear, in.
- $d$  = Distance from extreme compressive fiber to centroid of the pretensioning force, in.
- $f_y$  = Yield strength of steel reinforcement, ksi

Minimum area of horizontal shear reinforcement shall be

$$A_{vh} = 50 \frac{b_v s}{f_y} \quad (4.170)$$

where:

- $A_{vh}$  = Area of interface shear reinforcement, in.<sup>2</sup>
- $s$  = Center-to center spacing of interface shear reinforcement, in.

The spacing of tie reinforcement,  $s$ , shall not exceed four times the least web width of the girder nor 24 in.

The provisions for interface shear design specified by the LRFD Specifications are outlined as follows. For the strength limit state, the horizontal shear at a section shall be calculated using the following expression.

$$V_h = \frac{V_u}{d_e} \quad (4.171)$$

where:

- $V_h$  = Horizontal shear per unit length of the girder, kips
- $V_u$  = Factored shear force at specified section due to superimposed dead and live loads, kips
- $d_e$  = Distance between resultants of tensile and compressive forces, in.  
=  $(d_v - a/2)$
- $d_v$  = Distance between centroid of tension steel and top compression fiber, in.
- $a$  = Depth of equivalent stress block, in.

$$V_{n\ reqd} = V_h / \phi \quad (4.172)$$

where:

- $V_{n\ reqd}$  = Required nominal shear strength at the interface plane, kips
- $\phi$  = Resistance factor specified as 0.90 for shear in prestressed concrete members

The nominal shear resistance of the interface plane  $V_n$  is

$$V_n = c A_{cv} + \mu [A_{vf} f_y + P_c] \quad (4.173)$$

where:

- $c$  = Cohesion factor
- $\mu$  = Friction factor
- $A_{cv}$  = Area of concrete engaged in shear transfer, in<sup>2</sup>.
- $A_{vf}$  = Area of shear reinforcement crossing the shear plane, in<sup>2</sup>.
- $P_c$  = Permanent net compressive force normal to the shear plane, kips
- $f_y$  = Yield strength of shear reinforcement, ksi

The nominal shear resistance,  $V_n$  shall not be greater than the lesser of the following two values.

$$V_n \leq 0.2 f'_c A_{cv} \quad (4.174)$$

$$V_n \leq 0.8 A_{cv} \quad (4.175)$$

where:

$f'_c$  = The lower compressive strength at service of the two elements at the interface, ksi

For concrete placed against clean, hardened concrete and free of laitance, with the surface intentionally roughened to an amplitude of 0.25 in.:

$$c = 0.100 \text{ ksi}$$

$$\mu = 1.0\lambda$$

For concrete placed against clean, hardened concrete and free of laitance, but not intentionally roughened:

$$c = 0.075 \text{ ksi}$$

$$\mu = 0.6\lambda$$

$$\lambda = 1.0 \text{ for normal weight concrete}$$

The minimum interface shear reinforcement is determined as follows.

$$A_{vf} \geq (0.05b_v)/f_y \quad (4.176)$$

The above minimum shear reinforcement requirement may be waived if the value  $V_n/A_{cv} < 0.100 \text{ ksi}$



#### 4.4.8 Camber

The camber of pretensioned girders depends on several factors including modulus of elasticity of concrete, steel relaxation, dead load, concrete creep and shrinkage, erection loads, and live loads. The camber is a time-dependent quantity and it is difficult to provide an accurate measure of camber. The AASHTO Standard Specifications does not provide any specific method for the calculation of camber of pretensioned members. The LRFD Specifications provide guidelines in Article 5.7.3.6 for the calculation of effective moment of inertia for camber calculations. The previous research provides several different methodologies for the estimation of camber. The Hyperbolic Functions Method proposed by Sinno and Furr (1970) for the calculation of maximum camber of prestressed concrete is used in this parametric study. This is consistent with the TxDOT's prestressed bridge design software, PSTRS14 (TxDOT 2004). The methodology is outlined as follows.

Step 1: The total prestress,  $P$  after initial prestress loss has occurred is calculated using the following expression.

$$P = \frac{P_{si}}{\left(1 + pn + \frac{e_c^2 A_s n}{I}\right)} + \frac{M_D e_c A_s n}{I \left(1 + pn + \frac{e_c^2 A_s n}{I}\right)} \quad (4.177)$$

where:

- $P_{si}$  = Initial prestressing force, kips
- $I$  = Moment of inertia of non-composite section, in.<sup>4</sup>
- $e_c$  = Eccentricity of prestressing strands at the midspan, in.
- $M_D$  = Bending moment due to girder self weight at midspan, k-in.
- $p$  =  $A_s/A$
- $A_s$  = Area of prestressing strands, in.<sup>2</sup>
- $A$  = Area of cross section of girder, in.<sup>2</sup>
- $n$  = Modular ratio between girder concrete and prestressing steel.  
=  $E_p/E_c$

- $E_p$  = Modulus of elasticity of prestressing strands, ksi  
 $E_c$  = Modulus of elasticity of the girder concrete at release, ksi  
 $= 33(w_c)^{3/2} \sqrt{f'_c}$   
 $w_c$  = Unit weight of girder concrete, kcf  
 $f'_c$  = Girder concrete strength at service, ksi

The stress in concrete at the level of the centroid of the prestressing steel immediately after transfer is computed using the following expression.

$$f_{ci}^s = P \left( \frac{1}{A} + \frac{e_c^2}{I} \right) - f_c^s \quad (4.178)$$

where:

$$\begin{aligned}
 f_c^s &= \text{Concrete stress at the level of the centroid of the prestressing steel due to} \\
 &\quad \text{dead loads, ksi} \\
 &= \frac{M_D e_c}{I}
 \end{aligned}$$

Additional variables are the same as defined for Equation 4.178

The ultimate time dependent prestress loss is dependent on the ultimate creep and shrinkage strains. As the creep strains vary with the concrete stress, the following steps are used to evaluate the concrete stresses and adjust the strains to arrive at the ultimate prestress loss. It is assumed that the creep strain is proportional to the concrete stress, and the shrinkage stress is independent of concrete stress (Sinno et al. 1970).

Step 2: Initial estimate of the total strain at the level of centroid of prestressing steel, assuming constant sustained stress immediately after transfer, is calculated using the following expression.

$$\epsilon_{c1}^s = \epsilon_{cr}^\infty f_{ci}^s + \epsilon_{sh}^\infty \quad (4.179)$$

where:

$$\epsilon_{cr}^{\infty} = \text{Ultimate unit creep strain} = 0.00034 \text{ in./in.}$$

$$\epsilon_{sh}^{\infty} = \text{Ultimate unit shrinkage strain} = 0.000175 \text{ in./in.}$$

The above ultimate strain values were prescribed by Sinno et al. (1970) based on experimental results.

Step 3: The total strain obtained in Step 2 is adjusted by subtracting the elastic strain rebound as follows.

$$\epsilon_{c2}^s = \epsilon_{c1}^s - \epsilon_{c1}^s E_p \frac{A_s}{E_c} \left( \frac{1}{A} + \frac{e_c^2}{I} \right) \quad (4.180)$$

Step 4: The change in concrete stress at the level of centroid of prestressing steel due to strain adjustment is computed as follows.

$$\Delta f_c^s = \epsilon_{c2}^s E_p A_s \left( \frac{1}{A} + \frac{e_c^2}{I} \right) \quad (4.181)$$

Step 5: The total strain computed in Step 2 needs to be corrected for the change in the concrete stress due to creep and shrinkage strains.

$$\epsilon_{c4}^s = \epsilon_{cr}^{\infty} \left( f_{ci}^s - \frac{\Delta f_c^s}{2} \right) + \epsilon_{sh}^{\infty} \quad (4.182)$$

Step 6: The total strain obtained in Step 5 is adjusted by subtracting the elastic strain rebound as follows.

$$\epsilon_{c5}^s = \epsilon_{c4}^s - \epsilon_{c4}^s E_p \frac{A_s}{E_c} \left( \frac{1}{A} + \frac{e_c^2}{I} \right) \quad (4.183)$$

Sinno et al. (1970) recommends stopping the updating of stresses and adjustment process after Step 6. However, as the difference between the strains obtained in Steps 3 and 6 is not negligible, this process is carried on until the total strain value converges.

Step 7: The change in concrete stress at the level of centroid of prestressing steel is computed as follows.

$$\Delta f_{c1}^s = \epsilon_{c5}^s E_p A_s \left( \frac{1}{A} + \frac{e_c^2}{I} \right) \quad (4.184)$$

Step 8: The total strain computed in Step 5 needs to be corrected for the change in the concrete stress due to creep and shrinkage strains.

$$\epsilon_{c6}^s = \epsilon_{cr}^\infty \left( f_{ci}^s - \frac{\Delta f_c^s}{2} \right) + \epsilon_{sh}^\infty \quad (4.185)$$

Step 9: The total strain obtained in Step 8 is adjusted by subtracting the elastic strain rebound as follows.

$$\epsilon_{c7}^s = \epsilon_{c6}^s - \epsilon_{c6}^s E_p \frac{A_s}{E_c} \left( \frac{1}{A} + \frac{e_c^2}{I} \right) \quad (4.186)$$

Step 10: Steps 2 through 9 are repeated until the total strain value converges to a particular value. Then the initial prestress loss,  $PL_i$ , the final prestress loss,  $PL^\infty$ , and the total prestress loss,  $PL$ , are calculated using the following formulas

$$PL_i = \frac{P_{si} - P}{P_{si}} \quad (4.187)$$

$$PL^\infty = \frac{\epsilon_{c7}^s E_p A_s}{P_{si}} \quad (4.188)$$

$$\text{Total Prestress loss, } PL = PL_i + PL^\infty \quad (4.189)$$

Step 11: The initial deflection of the girder under self-weight,  $C_{DL}$ , is calculated using the elastic analysis as follows.

$$C_{DL} = \frac{5wL^4}{384E_{ci}I} \quad (4.190)$$

where:

- $C_{DL}$  = Initial deflection of the girder under self-weight, ft.
- $w$  = Self-weight of the girder, klf
- $L$  = Total girder length, ft.
- $E_{ci}$  = Modulus of elasticity of the girder concrete at release, ksi
- $I$  = Moment of inertia of the non-composite precast girder, in.<sup>4</sup>

Step 13: Step 12: Initial camber due to prestress is calculated using the moment area method. The following expression is obtained from the M/EI diagram to compute the camber resulting from the initial prestress.

$$C_{pi} = [0.5(P)(e_e)(0.5L)^2 + 0.5(P)(e_c - e_e)(0.67)(HD)^2 + 0.5P(e_c - e_e)(HD_{dis})(0.5L + HD)]/(E_{ci})(I) \quad (4.191)$$

where:

- $HD$  = Hold-down distance from the girder end, ft.
- $HD_{dis}$  = Hold-down distance from the center of the girder, ft.
- $e_e$  = Eccentricity of the prestressing strands at girder end, in.
- $e_c$  = Eccentricity of the prestressing strands at midspan, in.
- $L$  = Overall girder length, ft.

The net initial camber,  $C_i$ , is the difference between the upward camber due to initial prestressing and the downward deflection due to self-weight of the girder.

$$C_i = C_{pi} - C_{DL} \quad (4.192)$$

Step 14: The ultimate strain in the prestressing steel,  $\varepsilon_e^s$  and the ultimate time-dependent camber,  $C_t$ , is evaluated using the following expressions

$$\varepsilon_e^s = f_{ci}^s / E_c \quad (4.193)$$

$$C_t = C_i (1 - PL^\infty) \frac{\varepsilon_{cr}^\infty \left( f_{ci}^s - \frac{\Delta f_{cl}^s}{2} \right) + \varepsilon_e^s}{\varepsilon_e^s} \quad (4.194)$$

## 4.5 DESIGN PARAMETERS

### 4.5.1 General

The parameters considered for the parametric study are presented in Table 4.7. The span lengths were varied from 90 ft. to the maximum span possible limited by the release and service concrete strengths. TxDOT's procedures were used for optimizing the number of strands and concrete strengths.

**Table 4.7. Design Parameters**

| Parameter                               | Description / Selected Values   |
|---|---|
| Design Codes                            | AASHTO Standard Specifications, 17 <sup>th</sup> Edition (2002)<br>AASHTO LRFD Specifications, 3 <sup>rd</sup> Edition (2004)                 |
| Girder Section                          | Type C and AASHTO Type IV   |
| Girder Spacing (ft.)                    | 6'-0", 8'-0" and 8'-8"  |
| Spans                                   | 90 ft. to maximum span at 10 ft. intervals for Type IV girders<br>40 ft. to maximum span at 10 ft. intervals for Type C girders               |
| Strand Diameter (in.)                   | 0.5 and 0.6   |
| Concrete Strength at Release, $f_{ci}'$ | Varied from 4000 to 6750 psi for design with optimum number of strands  |
| Concrete Strength at Service, $f_c'$    | Varied from 5000 to 8500 psi for design with optimum number of strands ( $f_c'$ is increased up to 8750 psi for optimization on longer spans) |
| Skew Angle (degrees)                    | 0, 15, 30 and 60  |

The skew angles were varied for LRFD designs to investigate the impact of the skew which is introduced through the skew reduction factors for live load moments and skew correction factors for live load shears. The skew does not affect the designs based on AASHTO Standard Specifications as the distribution factors for live load are independent of the skew.

## **4.6 RESULTS AND SAMPLE OUTPUT**

### **4.6.1. General**

The parametric study was carried out for several possible cases satisfying the specified limits. The detailed results from the parametric study are presented in Sections 5 and 6 for AASHTO Type IV girders and for Type C girders, respectively. The output from the design program is presented in tabular and graphical formats and the impact on different design parameters is discussed. A sample output from the design program used in this study is presented in Table 4.8. This particular set of results is for AASHTO Type IV girders with 0.5 in. diameter strands using AASHTO Standard Specifications. The other parameters are included in the table.

**Table 4.8. Sample Output from Design Program.**

| Parameter  | Output 1 | Output 2 | Output 3 | Output 4 |
|--|----------|----------|----------|----------|
| Girder type (1-Type C and 2-Type IV)                         | 2        | 2        | 2        | 2        |
| Specifications (1-Std. and 2-LRFD)                           | 1        | 1        | 1        | 1        |
| Span, ft.  | 90       | 100      | 110      | 120      |
| Girder Spacing, ft.  | 8        | 8        | 8        | 8        |
| Max live load Moment (k-ft.)                                 | 1315.25  | 1494.38  | 1674.38  | 1854.38  |
| Live load shear at critical section (kips)                   | 62.32    | 63.30    | 64.10    | 64.77    |
| DFM  | 0.73     | 0.73     | 0.73     | 0.73     |
| DFV  | 0.73     | 0.73     | 0.73     | 0.73     |
| Distributed live load moment (k-ft.)                         | 1180.47  | 1329.87  | 1478.39  | 1625.47  |
| Distributed live load shear (kips)                           | 55.93    | 56.33    | 56.60    | 56.77    |
| Governing Moment (1-truck, 2-lane, 3-tandem)                 | 1        | 1        | 1        | 1        |
| Governing Shear (1-truck, 2-lane, 3-tandem)                  | 1        | 1        | 1        | 1        |
| Initial prestress loss, %                                    | 7.53     | 8.37     | 8.93     | 9.62     |
| Final prestress loss, %                                      | 19.48    | 22.72    | 25.22    | 28.57    |
| Required release concrete strength $f'_{ci}$ (psi)           | 4000.00  | 4478.26  | 5456.60  | 6538.36  |
| Required service concrete strength $f'_c$ (psi)              | 5000.00  | 5000.00  | 5583.89  | 7164.74  |
| Required number of strands                                   | 30       | 40       | 50       | 64       |
| Strength limit state moment $M_u$ (k-ft.)                    | 4932.01  | 5821.45  | 6769.39  | 7774.57  |
| Moment Resistance $M_r$ (k-ft.)                              | 5728.70  | 7398.09  | 8936.60  | 10836.04 |
| $M_u/M_r$  | 0.86     | 0.79     | 0.76     | 0.72     |
| Factored shear at critical section $V_u$ (kips)              | 222.17   | 235.11   | 247.77   | 260.22   |
| Concrete shear strength $V_c$ (kips)                         | 171.91   | 190.50   | 220.63   | 280.65   |
| Transverse shear reinf. area $A_v$ (in. <sup>2</sup> /ft.)   | 0.30     | 0.29     | 0.22     | 0.08     |
| Interface Shear $V_{nh}$ (kips)                              | 246.85   | 261.24   | 275.30   | 289.14   |
| Interface shear reinf. area $A_{vh}$ (in. <sup>2</sup> /ft.) | 0.20     | 0.20     | 0.20     | 0.20     |
| Camber (ft.)   | 0.16     | 0.24     | 0.32     | 0.40     |

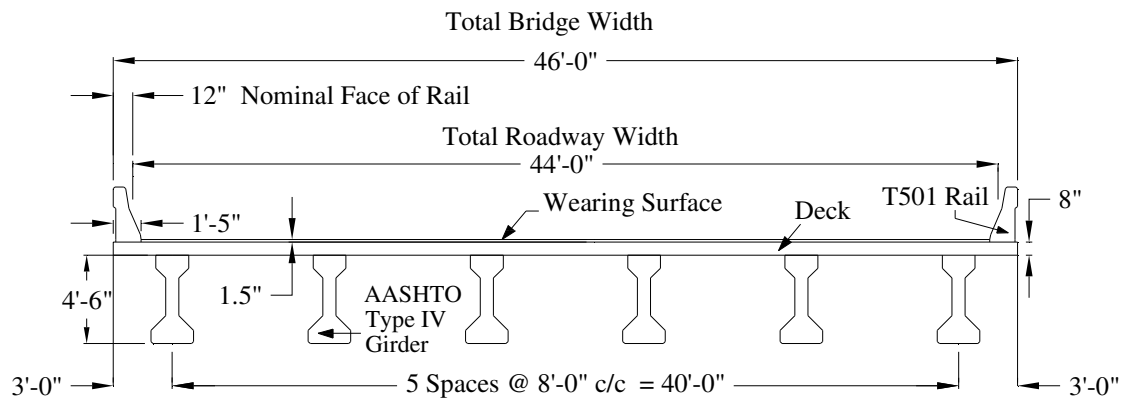
## 4.7 DETAILED DESIGN EXAMPLES

### 4.7.1 General

Two parallel detailed design examples were developed for an AASHTO Type IV girder bridge using the AASHTO Standard and LRFD Specifications. The cross section of the bridge considered for the detailed design examples is shown in Figure 4.15. The



detailed design examples are found in Appendix A.1 and Appendix A.2 for Standard and LRFD Specifications, respectively. The examples use the methodology described in Section 4.4.



**Figure 4.16. Bridge Cross Section.**

The following parameters were used for detailed design examples.

- span length = 110 ft.
- girder spacing = 8 ft.
- strand diameter = 0.5 in.
- deck slab thickness = 8 in.
- wearing surface thickness = 1.5 in.
- skew = 0°
- relative humidity = 60%.

The major differences occurring in the design due to differences in the specifications are highlighted. The detailed design examples are made comprehensive and in easy to follow format. The detailed examples are aimed to be a reference for bridge engineers and help in the transition from AASHTO Standard Specifications based design to AASHTO LRFD Specifications based design.

## 5. RESULTS FOR AASHTO TYPE IV GIRDERS

### 5.1 INTRODUCTION

A parametric study was conducted for AASHTO Type IV prestressed concrete bridge girders. Several cases were considered based on the parameters summarized in Table 5.1. The procedure outlined in Section 4 was employed to evaluate the impact of the AASHTO LRFD Specifications on the design of AASHTO Type IV bridge girders. The results obtained from the design program for designs based on both the Standard and LRFD Specifications were validated using TxDOT's PRSTRS14 (TxDOT 2004) bridge design software. TxDOT's procedures were used for optimizing the number of strands and concrete strengths. This section provides a summary of results of the parametric study for AASHTO Type IV bridge girders. The impact of LRFD specifications on various design results is discussed.

**Table 5.1. Design Parameters**

| Parameter                               | Description / Selected Values   |
|---|---|
| Design Codes                            | AASHTO Standard Specifications, 17th Edition (2002)<br>AASHTO LRFD Specifications, 3rd Edition (2004)   |
| Girder Spacing (ft.)                    | 6'-0", 8'-0" and 8'-8"  |
| Spans                                   | 90 ft. to maximum span at 10 ft. intervals  |
| Strand Diameter (in.)                   | 0.5 and 0.6   |
| Concrete Strength at Release, $f'_{ci}$ | Varied from 4000 to 6750 psi for design with optimum number of strands  |
| Concrete Strength at Service, $f'_c$    | Varied from 5000 to 8500 psi for design with optimum number of strands ( $f'_c$ may be increased up to 8750 psi for optimization on longer spans) |
| Skew Angle                              | 0°, 15°, 30° and 60°  |

## **5.2 IMPACT OF AASHTO LRFD SPECIFICATIONS**

The requirements for service load limit state design, flexural strength limit state design, transverse shear design, and interface shear design are evaluated in the parametric study. The designs based on LRFD Specifications are found to be conservative, in general as compared to the designs based on Standard Specifications. This conservatism is caused due to the increase in live load moments, more restrictive limits for service load design, and difference in the shear design approach. The effect of LRFD Specifications on the maximum allowable span length was investigated. The effect was found to be small.

The following sections provide the summary of differences observed in the designs based on Standard and LRFD Specifications. This includes the differences occurring in the undistributed and distributed live load moments, the distribution factors, the number of strands required, and required concrete strengths at release and at service. The differences observed in the flexural strength limit state design are provided in the following sections. The effect on camber is also evaluated and summarized. The differences in the transverse shear design and interface shear design are provided in Section 7.

## **5.3 IMPACT OF LRFD SPECIFICATIONS ON LIVE LOAD MOMENTS AND SHEARS**

### **5.3.1 General**

The Standard Specifications specify the live load to be taken as an HS-20 truck load, tandem load, or lane load, whichever produces the maximum effect at the section considered. The LRFD Specifications specifies a different live load model HL-93, which is a combination of the HS-20 truck and lane load, or tandem load and lane load, whichever produces maximum effect at the section of interest. The live load governing the moments and shears at the sections of interest for the cases considered in the parametric study was determined and are summarized below. The undistributed live load

moments at midspan and shears at critical section were calculated for each case and the representative differences are presented in this section.

There is a significant difference in the formulas for the distribution and impact factors specified by the Standard and the LRFD Specifications. The impact factors are applicable to truck, lane, and tandem loadings for designs based on Standard Specifications, whereas the LRFD Specifications does not require the lane load to be increased for the impact loading. The effect of the LRFD Specifications on the distribution and impact factors is evaluated and the results are summarized. The combined effect of the undistributed moments and shears and the distribution and impact factors on the distributed live load moments and shears was observed. The differences observed in the distributed live load moments at midspan and shears at the critical sections are presented below.

### **5.3.2 Governing Live Load for Moments and Shears**

The live load producing the maximum moment at mid-span and maximum shears at the critical section for shear is investigated. The critical section for shear in the designs based on Standard Specifications is taken as  $h/2$ , where  $h$  is the depth of composite section. For designs based on LRFD Specifications the critical section is calculated using an iterative process specified by the specifications. The governing live loads are summarized in the Tables 5.2 and 5.3.

**Table 5.2. Governing Live Load Moments at Midspan and Shears at Critical Section for Standard Specifications**

| <b>Strand Diameter (in.)</b> | <b>Girder Spacing (ft.)</b> | <b>Span (ft.)</b> | <b>Governing Live Load for Moment</b> | <b>Governing Live Load for Shear</b> |
|------------------------------|-----------------------------|-------------------|---------------------------------------|--------------------------------------|
| 0.5                          | 6                           | 90                | Truck Loading                         | Truck Loading                        |
|                              |                             | 100               |                                       |                                      |
|                              |                             | 110               |                                       |                                      |
|                              |                             | 120               |                                       |                                      |
|                              |                             | 130               |                                       |                                      |
|                              |                             | 136               |                                       | Lane Loading                         |
|                              | 8                           | 90                | Truck Loading                         | Truck Loading                        |
|                              |                             | 100               |                                       |                                      |
|                              |                             | 110               |                                       |                                      |
|                              |                             | 120               |                                       |                                      |
|                              |                             | 124               |                                       |                                      |
|                              | 8.67                        | 90                | Truck Loading                         | Truck Loading                        |
|                              |                             | 100               |                                       |                                      |
|                              |                             | 110               |                                       |                                      |
|                              |                             | 119               |                                       |                                      |
| 0.6                          | 6                           | 90                | Truck Loading                         | Truck Loading                        |
|                              |                             | 100               |                                       |                                      |
|                              |                             | 110               |                                       |                                      |
|                              |                             | 120               |                                       |                                      |
|                              |                             | 130               |                                       |                                      |
|                              |                             | 131               |                                       |                                      |
|                              | 8                           | 90                | Truck Loading                         | Truck Loading                        |
|                              |                             | 100               |                                       |                                      |
|                              |                             | 110               |                                       |                                      |
|                              |                             | 119               |                                       |                                      |
|                              | 8.67                        | 90                | Truck Loading                         | Truck Loading                        |
|                              |                             | 100               |                                       |                                      |
|                              |                             | 110               |                                       |                                      |
|                              |                             | 115               |                                       |                                      |

**Table 5.3. Governing Live Load Moments at Midspan and Shears at Critical Section for LRFD Specifications (Skew = 0°)**

| Strand Diameter (in.) | Girder Spacing (ft.) | Span (ft.) | Governing Live Load for Moment | Governing Live Load for Shear |
|-----------------------|----------------------|------------|--------------------------------|-------------------------------|
| 0.5                   | 6                    | 90         | Truck+Lane Loading             | Truck+Lane Loading            |
|                       |                      | 100        |                                |                               |
|                       |                      | 110        |                                |                               |
|                       |                      | 120        |                                |                               |
|                       |                      | 130        |                                |                               |
|                       |                      | 133        |                                |                               |
|                       | 8                    | 90         | Truck+Lane Loading             | Truck+Lane Loading            |
|                       |                      | 100        |                                |                               |
|                       |                      | 110        |                                |                               |
|                       |                      | 120        |                                |                               |
|                       | 8.67                 | 90         | Truck+Lane Loading             | Truck+Lane Loading            |
|                       |                      | 100        |                                |                               |
|                       |                      | 110        |                                |                               |
|                       |                      | 116        |                                |                               |
| 0.6                   | 6                    | 90         | Truck+Lane Loading             | Truck+Lane Loading            |
|                       |                      | 100        |                                |                               |
|                       |                      | 110        |                                |                               |
|                       |                      | 120        |                                |                               |
|                       |                      | 126        |                                |                               |
|                       | 8                    | 90         | Truck+Lane Loading             | Truck+Lane Loading            |
|                       |                      | 100        |                                |                               |
|                       |                      | 110        |                                |                               |
|                       |                      | 116        |                                |                               |
|                       | 8.67                 | 90         | Truck+Lane Loading             | Truck+Lane Loading            |
|                       |                      | 100        |                                |                               |
|                       |                      | 110        |                                |                               |
|                       |                      | 113        |                                |                               |

It was observed that for Standard Specifications based designs, HS-20 Truck loading always governs the moments at mid-span and shears at critical sections except for 136 ft. span case. For designs based on LRFD Specifications, combination of Truck and Lane loading governs for all the cases.

### 5.3.3 Undistributed Live Load Moments and Shears

The difference in the live loads specified by the Standard and the LRFD Specifications effects the undistributed live load moments and shears. Skew and strand diameter has no effect on the undistributed live load moments or shears therefore results for cases with skew angle  $0^\circ$  and strand diameter 0.5 in. are compared in Table 5.4. The undistributed live load moments are observed to be increasing in the range of 48 percent to 65 percent for 6 ft. girder spacing when live loads based on LRFD Specifications are used as compared to the Standard Specifications. This increase was in the range of 48 percent to 61 percent for 8 ft. girder spacing and 48 percent to 56 percent for a 8.67 ft. girder spacing.

A significant increase was observed in the undistributed shears at critical section. The increase was found to be in the range of 35 percent to 54 percent for 6 ft. girder spacing when LRFD Specifications are used as compared to Standard Specifications. This increase was found to be in the range of 35 percent to 50 percent for 8 ft. girder spacing and 35 percent to 45 percent for 8.67 ft. girder spacing. This increase can be attributed the change in live load and also the shifting of critical section. The critical section for shear is specified by Standard specifications as  $h/2$ , where  $h$  is the depth of composite section. The LRFD Specifications requires the critical section to be calculated using an iterative process as discussed in Section 4. The difference between the undistributed moments and shears based on Standard and LRFD Specifications is found to be increasing with the increase in span length.

**Table 5.4. Comparison of Undistributed Midspan Live Load Moments and Shears at Critical Section (Skew = 0°, Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Undistributed Moment (k-ft.) |        |                      | Undistributed Shear (kips) |       |                     |
|----------------------|------------|------------------------------|--------|----------------------|----------------------------|-------|---------------------|
|                      |            | Standard                     | LRFD   | Difference k-ft. (%) | Standard                   | LRFD  | Difference kips (%) |
| 6                    | 90         | 1315.2                       | 1943.0 | 627.8 (47.7)         | 62.3                       | 83.7  | 21.4 (34.4)         |
|                      | 100        | 1494.4                       | 2271.9 | 777.5 (52.0)         | 63.3                       | 88.2  | 24.9 (39.3)         |
|                      | 110        | 1674.4                       | 2617.6 | 943.2 (56.3)         | 64.1                       | 92.6  | 28.5 (44.4)         |
|                      | 120        | 1854.4                       | 2979.3 | 1125.0 (60.7)        | 64.8                       | 96.6  | 31.9 (49.2)         |
|                      | 130        | 2034.4                       | 3357.1 | 1322.7 (65.0)        | 65.3                       | 100.5 | 35.2 (53.9)         |
|                      | 133        | -                            | 3473.5 | -                    | -                          | 102.5 | -                   |
|                      | 136        | 2142.4                       | -      | -                    | 66.9                       | -     | -                   |
| 8                    | 90         | 1315.2                       | 1943.0 | 627.8 (47.7)         | 62.3                       | 84.0  | 21.7 (34.8)         |
|                      | 100        | 1494.4                       | 2271.9 | 777.5 (52.0)         | 63.3                       | 88.6  | 25.3 (40.0)         |
|                      | 110        | 1674.4                       | 2617.6 | 943.2 (56.3)         | 64.1                       | 92.6  | 28.5 (44.4)         |
|                      | 120        | 1854.4                       | 2979.3 | 1125.0 (60.7)        | 64.8                       | 96.9  | 32.2 (49.7)         |
|                      | 124        | 1926.4                       | -      | -                    | 65.0                       | -     | -                   |
| 8.67                 | 90         | 1315.2                       | 1943.0 | 627.8 (47.7)         | 62.3                       | 84.1  | 21.8 (34.9)         |
|                      | 100        | 1494.4                       | 2271.9 | 777.5 (52.0)         | 63.3                       | 88.7  | 25.4 (40.1)         |
|                      | 110        | 1674.4                       | 2617.6 | 943.2 (56.3)         | 64.1                       | 92.6  | 28.5 (44.5)         |
|                      | 116        | -                            | 2832.7 | -                    | -                          | 95.4  | -                   |
|                      | 119        | 1836.4                       | -      | -                    | 64.7                       | -     | -                   |

### 5.3.4 Impact Factors

The AASHTO Standard and LRFD Specifications require that live load moments and shears be increased for impact or dynamic loading. The Standard Specifications specifies impact factors that decrease with an increase in span length, whereas the LRFD Specifications specify a constant value of dynamic loading as 33 percent of the undistributed live load moment or shear. For fatigue load moment, the LRFD Specifications specify the impact loading to be 15 percent of the undistributed live load fatigue moment. The fatigue moments are used to check the fatigue limit state required by the LRFD Specifications. The LRFD Specifications do not require the lane load moments and shears to be increased for impact loading.

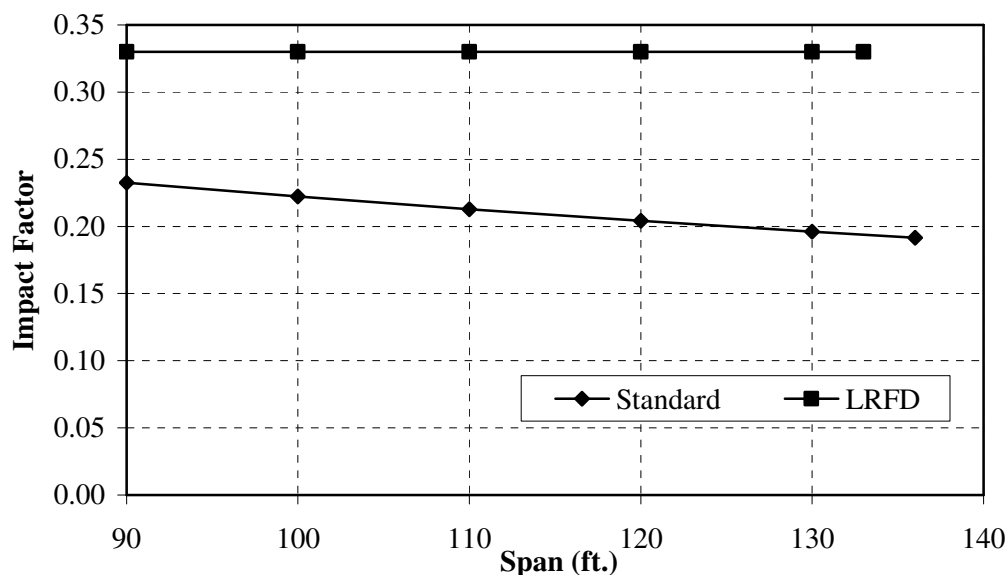


A summary of impact factors and the percent difference relative to Standard value is provided in Table 5.5. The skew angle and strand diameter do not affect the impact factor, hence only the cases with skew angle  $0^\circ$  and strand diameter 0.5 in. are presented. It was observed that the LRFD Specifications provides a larger estimate of dynamic loading as compared to the Standard Specifications. This difference increases with increasing span length. The increase in the impact factor is in the range of 42 percent to 68 percent of the impact factors specified by Standard Specifications. This essentially increases the distributed live load moments for the designs based on LRFD Specifications as compared to the Standard Specifications.

**Table 5.5. Comparison of Live Load Impact Factors  
(Skew =  $0^\circ$ , Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Impact Factor |      | Difference (%) |
|----------------------|------------|---------------|------|----------------|
|                      |            | Standard      | LRFD |                |
| 6                    | 90         | 0.23          | 0.33 | 41.9           |
|                      | 100        | 0.22          | 0.33 | 48.5           |
|                      | 110        | 0.21          | 0.33 | 55.1           |
|                      | 120        | 0.20          | 0.33 | 61.7           |
|                      | 130        | 0.20          | 0.33 | 68.3           |
|                      | 133        | -             | 0.33 | -              |
|                      | 136        | 0.19          | -    | -              |
| 8                    | 90         | 0.23          | 0.33 | 41.9           |
|                      | 100        | 0.22          | 0.33 | 48.5           |
|                      | 110        | 0.21          | 0.33 | 55.1           |
|                      | 120        | 0.20          | 0.33 | 61.7           |
|                      | 124        | 0.20          | -    | -              |
| 8.67                 | 90         | 0.23          | 0.33 | 41.9           |
|                      | 100        | 0.22          | 0.33 | 48.5           |
|                      | 110        | 0.21          | 0.33 | 55.1           |
|                      | 116        | -             | 0.33 | -              |
|                      | 119        | 0.20          | -    | -              |

Figure 5.1. illustrates the impact of the LRFD Specifications on the dynamic load (impact) factors for a 6 ft. girder spacing. The same trend was observed for girder spacings of 8 ft. and 8.67 ft.



**Figure 5.1. Impact Factors for AASHTO Standard vs. AASHTO LRFD Specifications (Girder Spacing = 6 ft., Skew = 0°, Strand Diameter = 0.5 in)**

### 5.3.5 Live Load Distribution Factors

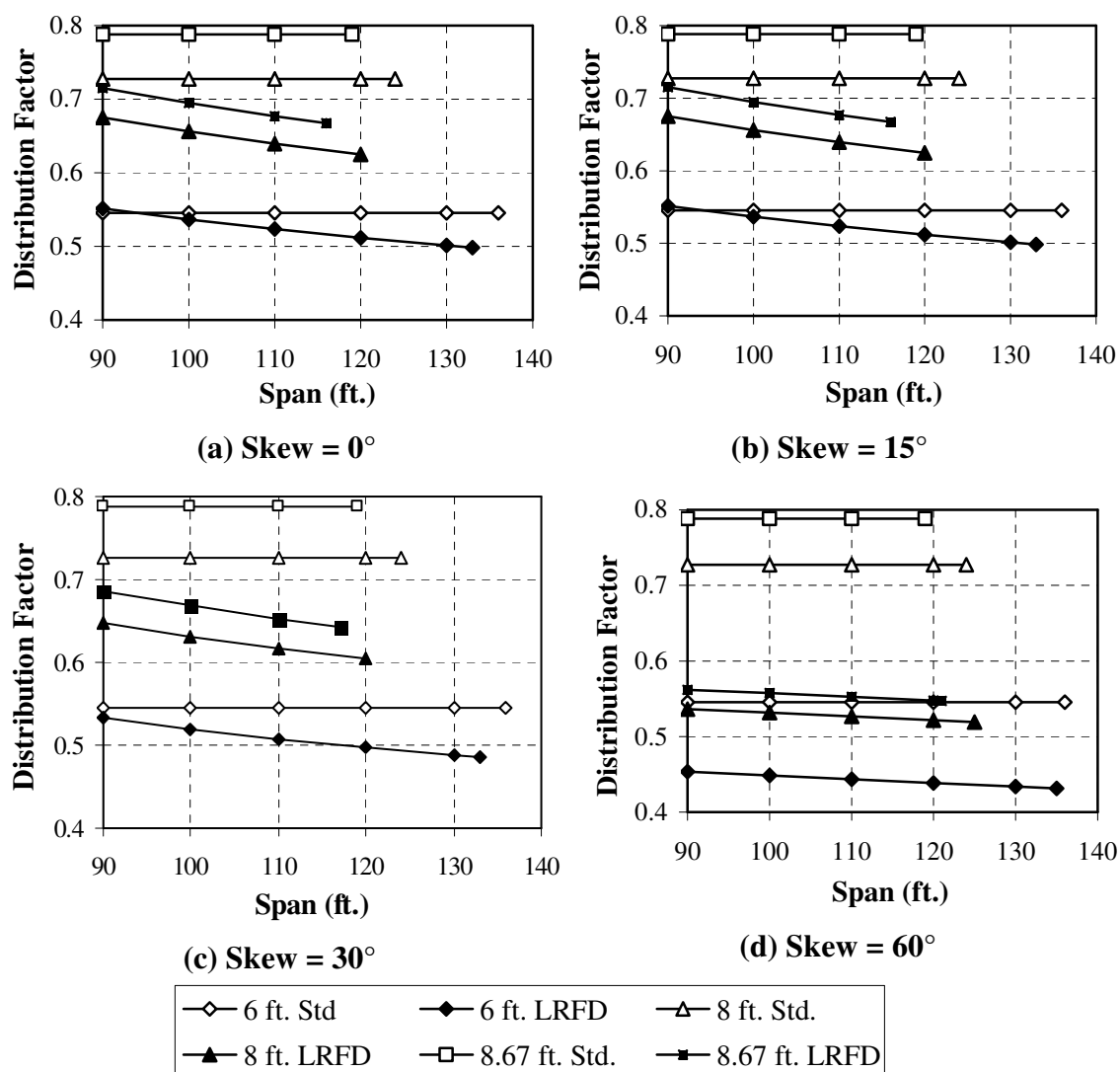
The live load moments and shears, including the dynamic (impact) load effect are distributed to the individual girders. The Standard Specifications provide a simple formula for moment distribution factor (DF) as  $S/11$  for prestressed concrete girder bridges, where  $S$  is the girder spacing in ft. The same DF is used for the distribution of live load shear to the girders. The LRFD Specifications provides more complex formulas for the distribution of live load moments and shears to individual girders. The effects of beam and slab stiffness are incorporated into these formulas. The LRFD Specifications requires the DFs for moment to be reduced and DFs for shear to be corrected for skewed bridges. Table 5.6 compares the live load moment DFs for the Standard and LRFD Specifications.

**Table 5.6. Comparison of Live Load Moment DFs (DFM)  
(Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. DFM | LRFD      |         |            |         |            |         |            |         |
|----------------------|------------|----------|-----------|---------|------------|---------|------------|---------|------------|---------|
|                      |            |          | skew = 0° |         | skew = 15° |         | skew = 30° |         | skew = 60° |         |
|                      |            |          | DFM       | Diff. % | DFM        | Diff. % | DFM        | Diff. % | DFM        | Diff. % |
| 6                    | 90         | 0.545    | 0.552     | 1.1     | 0.552      | 1.1     | 0.533      | -2.3    | 0.453      | -16.9   |
|                      | 100        | 0.545    | 0.537     | -1.6    | 0.537      | -1.6    | 0.520      | -4.7    | 0.448      | -17.8   |
|                      | 110        | 0.545    | 0.523     | -4.0    | 0.523      | -4.0    | 0.508      | -6.9    | 0.443      | -18.7   |
|                      | 120        | 0.545    | 0.512     | -6.2    | 0.512      | -6.2    | 0.498      | -8.8    | 0.439      | -19.6   |
|                      | 130        | 0.545    | 0.501     | -8.1    | 0.501      | -8.1    | 0.488      | -10.5   | 0.434      | -20.5   |
|                      | 133        | -        | 0.498     | -       | 0.498      | -       | 0.486      | -       | -          | -       |
|                      | 135        | -        | -         | -       | -          | -       | -          | -       | 0.431      | -       |
|                      | 136        | 0.545    | -         | -       | -          | -       | -          | -       | -          | -       |
| 8                    | 90         | 0.727    | 0.675     | -7.2    | 0.675      | -7.2    | 0.648      | -10.9   | 0.536      | -26.3   |
|                      | 100        | 0.727    | 0.656     | -9.8    | 0.656      | -9.8    | 0.632      | -13.1   | 0.532      | -26.9   |
|                      | 110        | 0.727    | 0.639     | -12.1   | 0.639      | -12.1   | 0.618      | -15.1   | 0.527      | -27.6   |
|                      | 120        | 0.727    | 0.625     | -14.1   | 0.625      | -14.1   | 0.605      | -16.8   | 0.522      | -28.3   |
|                      | 124        | 0.727    | -         | -       | -          | -       | -          | -       | -          | -       |
|                      | 125        | -        | -         | -       | -          | -       | -          | -       | 0.519      | -       |
| 8.67                 | 90         | 0.788    | 0.715     | -9.3    | 0.715      | -9.3    | 0.685      | -13.0   | 0.562      | -28.7   |
|                      | 100        | 0.788    | 0.695     | -11.9   | 0.695      | -11.9   | 0.668      | -15.2   | 0.557      | -29.3   |
|                      | 110        | 0.788    | 0.677     | -14.1   | 0.677      | -14.1   | 0.653      | -17.1   | 0.553      | -29.9   |
|                      | 116        | -        | 0.667     | -       | 0.667      | -       | -          | -       | -          | -       |
|                      | 117        | -        | -         | -       | -          | -       | 0.643      | -       | -          | -       |
|                      | 119        | 0.788    | -         | -       | -          | -       | -          | -       | -          | -       |
|                      | 120        | -        | -         | -       | -          | -       | -          | -       | 0.548      | -       |
|                      | 121        | -        | -         | -       | -          | -       | -          | -       | 0.547      | -       |

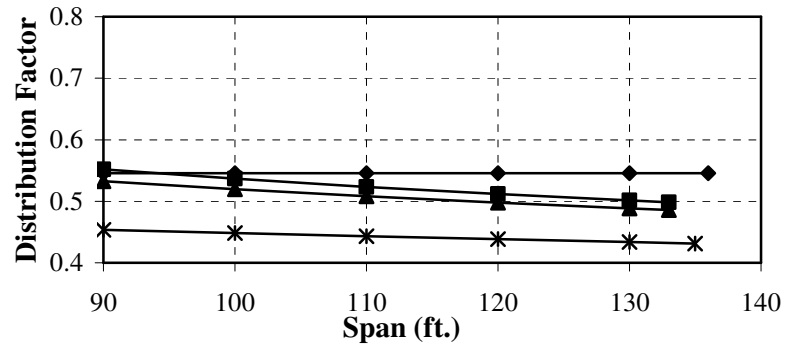
It was observed that the live load moment DFs given by the LRFD Specifications are typically smaller as compared to those for the Standard Specifications. The difference increases with an increase in span length because the LRFD DFs decrease with an increase in the span while span length has no effect on the Standard DFs. The moment DFs increase with increase in girder spacing for both the AASHTO Standard and LRFD Specifications. In addition, the difference between the DFs increased for larger girder spacings. The LRFD live load moment DFs are the same for 0° and 15°

skews, but there is a significant change when the skew angles are  $30^\circ$  and  $60^\circ$ . It was observed that increase in skew angles beyond  $30^\circ$  decreases the moment DFs significantly for AASHTO Type IV girder bridges. The maximum difference between the Standard and LRFD DFs was found to be 8 percent for 6 ft. girder spacing, 14 percent for 8 ft and 8.67 ft. girder spacing for the skew angle of  $0^\circ$ . This difference increased to 21 percent for 6 ft., 28 percent for 8 ft. and 30 percent for 8.67 ft. girder spacing for a skew angle of  $60^\circ$ .

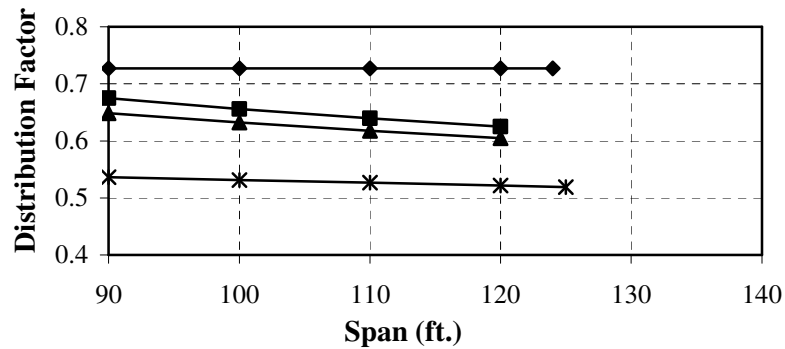


**Figure 5.2. Comparison of Live Load Moment DFs by Skew Angle (Strand Dia. = 0.5 in.)**

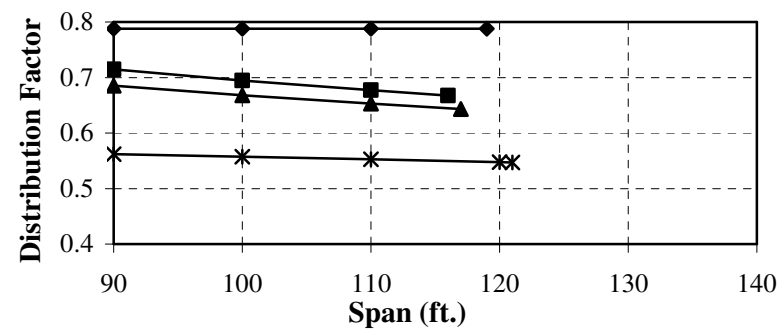
Figure 5.2 shows the effect of girder spacing and span length on the moment DFs for skew angles 0°, 15°, 30° and 60°. Figure 5.3 shows the effect of skew on the moment DFs for 6 ft., 8 ft. and 8.67 ft. girder spacing.



(a) Girder Spacing = 6 ft.



(b) Girder Spacing = 8 ft.



(c) Girder Spacing = 8.67 ft.



Figure 5.3. Comparison of Live Load Moment DFs by Girder Spacing (Strand Dia. = 0.5 in.)

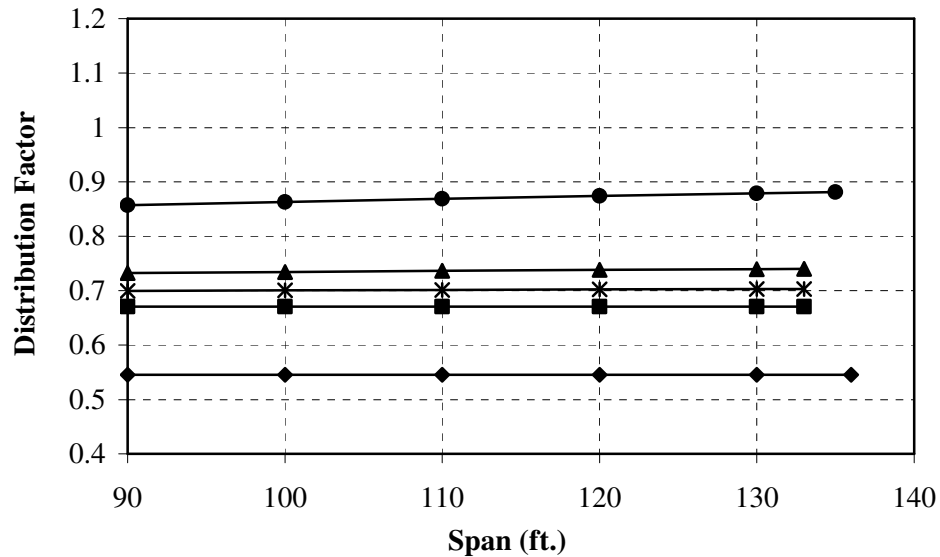
Table 5.7 and Figure 5.4 provide a summary of shear DFs for the parametric study with AASHTO Type IV girders. The strand diameter does not affect the DFs for shear.

**Table 5.7. Comparison of Live Load Shear DFs (DFV)  
(Strand Diameter = 0.5 in.)**

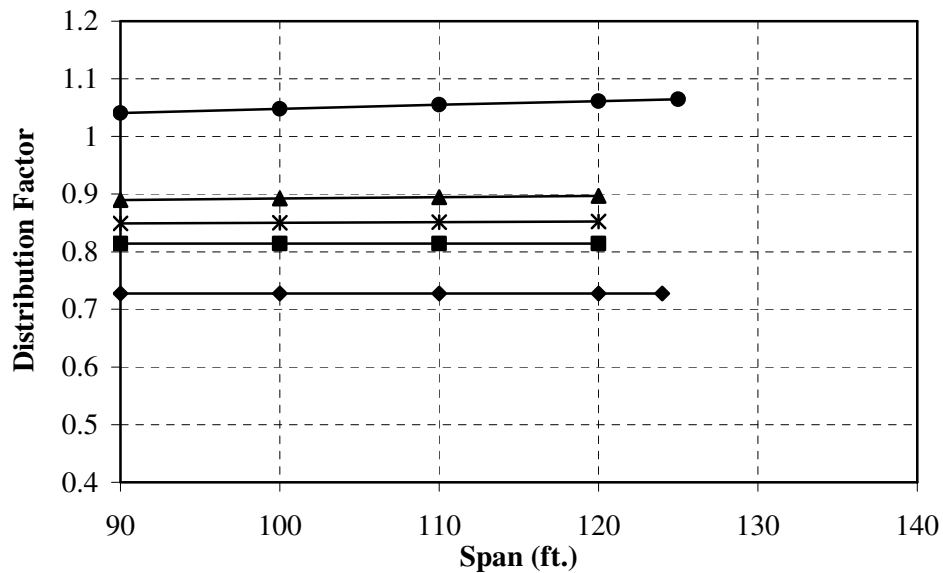
| Girder Spacing (ft.) | Span (ft.) | Std. DFV | LRFD      |         |            |         |            |         |            |         |
|----------------------|------------|----------|-----------|---------|------------|---------|------------|---------|------------|---------|
|                      |            |          | skew = 0° |         | skew = 15° |         | Skew = 30° |         | skew = 60° |         |
|                      |            |          | DFV       | Diff. % | DFV        | Diff. % | DFV        | Diff. % | DFV        | Diff. % |
| 6                    | 90         | 0.545    | 0.671     | 22.9    | 0.699      | 28.2    | 0.733      | 34.3    | 0.857      | 57.1    |
|                      | 100        | 0.545    | 0.671     | 22.9    | 0.700      | 28.4    | 0.735      | 34.7    | 0.863      | 58.3    |
|                      | 110        | 0.545    | 0.671     | 22.9    | 0.701      | 28.6    | 0.737      | 35.1    | 0.869      | 59.3    |
|                      | 120        | 0.545    | 0.671     | 22.9    | 0.702      | 28.7    | 0.738      | 35.4    | 0.874      | 60.3    |
|                      | 130        | 0.545    | 0.671     | 22.9    | 0.703      | 28.9    | 0.740      | 35.7    | 0.879      | 61.2    |
|                      | 133        | -        | 0.671     | -       | 0.703      | -       | 0.741      | -       | -          | -       |
|                      | 135        | -        | -         | -       | -          | -       | -          | -       | 0.882      | -       |
|                      | 136        | 0.545    | -         | -       | -          | -       | -          | -       | -          | -       |
| 8                    | 90         | 0.727    | 0.814     | 12.0    | 0.849      | 16.8    | 0.890      | 22.4    | 1.041      | 43.1    |
|                      | 100        | 0.727    | 0.814     | 12.0    | 0.851      | 17.0    | 0.892      | 22.7    | 1.048      | 44.1    |
|                      | 110        | 0.727    | 0.814     | 12.0    | 0.852      | 17.1    | 0.895      | 23.0    | 1.055      | 45.1    |
|                      | 120        | 0.727    | 0.814     | 12.0    | 0.853      | 17.2    | 0.897      | 23.3    | 1.062      | 46.0    |
|                      | 124        | 0.727    | -         | -       | -          | -       | -          | -       | -          | -       |
|                      | 125        | -        | -         | -       | -          | -       | -          | -       | 1.065      | -       |
| 8.67                 | 90         | 0.788    | 0.861     | 9.3     | 0.898      | 14.0    | 0.941      | 19.4    | 1.101      | 39.6    |
|                      | 100        | 0.788    | 0.861     | 9.3     | 0.899      | 14.1    | 0.944      | 19.7    | 1.108      | 40.6    |
|                      | 110        | 0.788    | 0.861     | 9.3     | 0.901      | 14.3    | 0.946      | 20.0    | 1.116      | 41.6    |
|                      | 116        | -        | 0.861     | -       | 0.901      | -       | -          | -       | -          | -       |
|                      | 117        | -        | -         | -       | -          | -       | 0.948      | -       | -          | -       |
|                      | 119        | 0.788    | -         | -       | -          | -       | -          | -       | -          | -       |
|                      | 120        | -        | -         | -       | -          | -       | -          | -       | 1.123      | -       |
|                      | 121        | -        | -         | -       | -          | -       | -          | -       | 1.123      | -       |

The LRFD live load shear DFs specified by LRFD Specifications are larger as compared to the Standard Specifications. The DFs increases with an increase in girder spacing for both specifications and the LRFD DFs approach Standard DFs as the girder spacing is increased. The span length and skew angle has no impact on the shear

distribution factors specified by Standard Specifications. The maximum difference in the shear distribution factors is found to be 61 percent for 6 ft. spacing, 46 percent for 8 ft. spacing and 42 percent for the 8.67 ft. spacing.

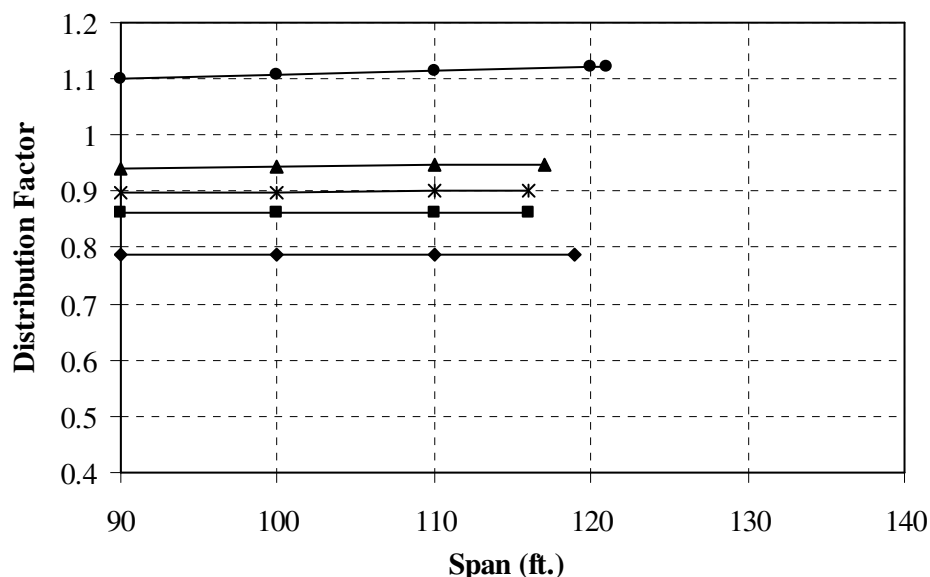


(a) Girder Spacing = 6 ft.

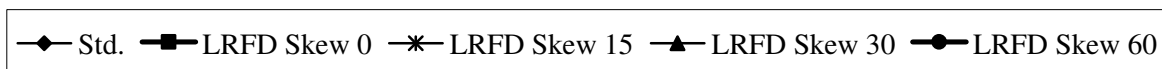


(b) Girder Spacing = 8 ft.

**Figure 5.4. Comparison of Live Load Shear DFs  
(Strand Dia. = 0.5 in.)**



(c) Girder Spacing = 8.67 ft.



**Figure 5.4. (Cont.) Comparison of Live Load Shear DFs  
(Strand Dia. = 0.5 in.)**

### 5.3.6 Distributed Live Load Moments and Shears

The combined effect of the impact and distribution factors on the live load moment and shears is presented in this section. The distributed live load moments are compared in Table 5.8. The distributed live load moments are the same for 0° and 15° skew angles for LRFD Specifications because the distribution factors for these two skews are identical. The distributed live load moments were found to be significantly larger than those for the Standard Specifications. The distributed live load moments increase in the range of 48 percent to 52 percent for 6 ft. girder spacing when the skew angle is 0°. As the girder spacing increases the difference between the distributed live load moments decreases. The LRFD moments were found to be in the range of 36 percent to 38 percent larger for 8 ft. and 33 percent to 38 percent larger for 8.67 ft. girder spacing when the skew angle is 0°.



The increase in skew angle for and beyond 30° results in decrease in distribution factors. This causes the live load moments to decrease. The difference between the live load moments for Standard and LRFD Specifications was found to be in the range of 22 percent to 32 percent for 6 ft. girder spacing when skew angle is 60°. This difference reduces to the range of 8 percent to 15 percent and 4 percent to 9 percent for 8 ft. and 8.67 ft. girder spacing respectively. The increase in span length increases the difference between the live load moments specified by the two codes.

**Table 5.8. Comparison of Distributed Midspan Live Load Moments (LL Mom.) for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**

[illegible]

The distributed shear force at the critical section due to live load is found to be increasing significantly when LRFD Specifications are used. The increase in the shear force can be attributed to the increase in the undistributed shear force due to HL93 loading and the increase in distribution factors. The shear force at the critical section for LRFD Specifications is found to be increasing in the range of 124 percent to 160 percent for 6 ft. girder spacing as compared to the Standard specifications. The increase was found to be in the range of 104 percent to 129 percent for 8 ft. and 99 percent to 115 percent for 8.67 ft. girder spacing. The results are presented in Table 5.9.

**Table 5.9. Comparison of Distributed Live Load Shear at Critical Section for Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**

[illegible]

## **5.4 IMPACT OF AASHTO LRFD SPECIFICATIONS ON SERVICE LOAD DESIGN**

### **5.4.1 General**

The impact of LRFD Specifications on the service load design results is discussed in this section. The effect on prestress losses, required number of strands and the required concrete strengths at service and at release is discussed. The increase in the live load moment and the change in equations for prestress loss calculations specified by AASHTO LRFD Specifications results in different service load design requirements. The change in the service load combination and allowable stress limits also affects the design. Generally the design requirements for LRFD Specifications were found to be conservative as compared to Standard Specifications.

### **5.4.2 Impact on Prestress Losses**

The loss in prestress occurs mainly from four sources viz. shrinkage of concrete, relaxation of steel, elastic shortening of steel and creep of concrete. These losses are categorized into initial prestress loss and final prestress loss. The initial prestress loss occurs due to initial relaxation of steel and elastic shortening of prestressing strands. The final loss occurs due to final relaxation, creep and shrinkage of concrete. Total prestress loss is the combination of initial and final loss.

#### *5.4.2.1 Prestress Loss Due to Elastic Shortening of Steel*

The loss of prestress due to elastic shortening of steel is dependent on the modulus of the prestressing strands, modulus of concrete at release and the number of prestressing strands. The modulus of the elasticity of prestressing strands is specified by Standard specifications as 28000 ksi and LRFD Specifications as 28500 ksi. The modulus of the concrete depends on the concrete strength at release. The required concrete strength at release is different for Standard and LRFD Specifications. The combined effect of these parameters results in a non-uniform trend. The prestress loss

due to elastic shortening was found to be increasing for LRFD Specifications based design, except for a few cases when skew angle was 60°.

**Table 5.10. Comparison of Prestress Loss Due to Elastic Shortening (ES) for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. ES (ksi) | LRFD     |         |          |         |          |         |          |         |
|----------------------|------------|---------------|----------|---------|----------|---------|----------|---------|----------|---------|
|                      |            |               | Skew 0°  |         | Skew 15° |         | Skew 30° |         | Skew 60° |         |
|                      |            |               | ES (ksi) | Diff. % | ES (ksi) | Diff. % | ES (ksi) | Diff. % | ES (ksi) | Diff. % |
| 6                    | 90         | 11.33         | 12.95    | 14.3    | 12.95    | 14.3    | 12.95    | 14.3    | 11.49    | 1.4     |
|                      | 100        | 13.28         | 14.05    | 5.8     | 14.05    | 5.8     | 13.45    | 1.3     | 12.81    | -3.5    |
|                      | 110        | 14.16         | 15.33    | 8.3     | 15.33    | 8.3     | 15.33    | 8.2     | 14.84    | 4.8     |
|                      | 120        | 15.62         | 16.97    | 8.6     | 16.97    | 8.6     | 16.60    | 6.2     | 16.20    | 3.7     |
|                      | 130        | 16.91         | 18.84    | 11.4    | 18.84    | 11.4    | 18.44    | 9.0     | 17.68    | 4.5     |
|                      | 133        | -             | 19.28    | -       | 19.28    | -       | 19.28    | -       | -        | -       |
|                      | 135        | -             | -        | -       | -        | -       | -        | -       | 18.87    | -       |
|                      | 136        | 18.31         | -        | -       | -        | -       | -        | -       | -        | -       |
| 8                    | 90         | 14.02         | 14.78    | 5.4     | 14.78    | 5.4     | 14.80    | 5.6     | 13.59    | -3.0    |
|                      | 100        | 15.95         | 16.58    | 4.0     | 16.58    | 4.0     | 16.14    | 1.2     | 15.68    | -1.7    |
|                      | 110        | 17.24         | 18.13    | 5.2     | 18.13    | 5.2     | 17.79    | 3.2     | 17.43    | 1.1     |
|                      | 120        | 18.88         | 20.11    | 6.5     | 20.11    | 6.5     | 19.93    | 5.6     | 19.07    | 1.0     |
|                      | 124        | 19.91         | -        | -       | -        | -       | -        | -       | -        | -       |
|                      | 125        | -             | -        | -       | -        | -       | -        | -       | 20.11    | -       |
| 8.67                 | 90         | 14.59         | 15.33    | 5.1     | 15.33    | 5.1     | 15.36    | 5.2     | 14.20    | -2.7    |
|                      | 100        | 16.38         | 17.01    | 3.8     | 17.01    | 3.8     | 17.00    | 3.8     | 16.14    | -1.5    |
|                      | 110        | 17.93         | 19.08    | 6.4     | 19.08    | 6.4     | 18.80    | 4.9     | 17.78    | -0.8    |
|                      | 116        | -             | 20.14    | -       | 20.14    | -       | -        | -       | -        | -       |
|                      | 117        | -             | -        | -       | -        | -       | 20.10    | -       | -        | -       |
|                      | 119        | 19.90         | -        | -       | -        | -       | -        | -       | -        | -       |
|                      | 120        | -             | -        | -       | -        | -       | -        | -       | 19.93    | -       |
|                      | 121        | -             | -        | -       | -        | -       | -        | -       | 20.07    | -       |

The increase in girder spacing resulted in a decrease in the difference between the loss calculated using the two specifications. The skew angle does not have a well defined effect on the loss but for skew angle of 60° the loss is found to be decreasing. This can be attributed to the decrease in the live load moments thereby decreasing the

number of prestressing strands and consequently the stress in the concrete. Similar trends were observed for 0.5 in and 0.6 in. diameter strands. The results for 0.5 in. diameter strand are presented in Table 5.10.

#### *5.4.2.2 Prestress Loss Due to Initial Steel Relaxation*

The loss in prestress due to initial relaxation of steel is specified by LRFD Specifications as a function of time, jacking and the yield stress of the prestressing strands. The time for release of prestress is taken as 24 hours. This provides a constant estimate of initial relaxation loss as 1.98 ksi. This does not have any effect of skew and strand diameter. The Standard Specifications do not specify a particular formula to evaluate the initial relaxation loss. Following the TxDOT practices (TxDOT 2001) the initial relaxation loss is taken as half the total relaxation loss.

It was observed that the prestress loss due to relaxation calculated in accordance with LRFD Specifications yields a conservative estimate. This conservatism for 0.5 in. diameter strands is in the range of 36 percent to 148 percent for 6 ft., 62 percent to 223 percent for 8 ft. and 70 percent to 168 percent for 8.67 ft. girder spacing. The increase in the initial relaxation loss was found to be in the range of 48 percent to 116 percent for 6 ft., 78 percent to 143 percent for 8 ft. and 72 percent to 168 percent for 8.67 ft. girder spacing when 0.6 in. diameter strands were used. The conservatism is found to be increasing with the increase in span and also with the increase in girder spacing. Table 5.11 shows the results for 0.5 in. strand diameter and the results for 0.6 in. strand diameter are presented in Table 5.12. The cases with only skew angle  $0^\circ$  are compared as the skew angle has no effect on the initial relaxation loss.

**Table 5.11. Comparison of Prestress Loss due to Initial Steel Relaxation for AASHTO Standard and LRFD Specifications (Skew = 0°, Strand Dia. = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Initial Relaxation Loss (ksi) |      | Difference % |
|----------------------|------------|-------------------------------|------|--------------|
|                      |            | Standard                      | LRFD |              |
| 6                    | 90         | 1.45                          | 1.98 | 36.1         |
|                      | 100        | 1.26                          | 1.98 | 57.5         |
|                      | 110        | 1.17                          | 1.98 | 69.3         |
|                      | 120        | 0.99                          | 1.98 | 100.8        |
|                      | 130        | 0.80                          | 1.98 | 147.5        |
|                      | 133        | -                             | 1.98 | -            |
|                      | 136        | 0.65                          | -    | -            |
| 8                    | 90         | 1.22                          | 1.98 | 61.6         |
|                      | 100        | 1.00                          | 1.98 | 97.8         |
|                      | 110        | 0.83                          | 1.98 | 137.4        |
|                      | 120        | 0.61                          | 1.98 | 223.4        |
|                      | 124        | 0.48                          | -    | -            |
| 8.67                 | 90         | 1.16                          | 1.98 | 70.4         |
|                      | 100        | 0.95                          | 1.98 | 108.1        |
|                      | 110        | 0.74                          | 1.98 | 167.4        |
|                      | 116        | -                             | 1.98 | -            |
|                      | 119        | 0.49                          | -    | -            |

**Table 5.12. Comparison of Prestress Loss due to Initial Steel Relaxation for Standard and LRFD Specifications (Skew = 0°, Strand Dia. = 0.6 in.)**

| Girder Spacing (ft.) | Span (ft.) | Initial Relaxation Loss (ksi) |      | Difference % |
|----------------------|------------|-------------------------------|------|--------------|
|                      |            | Standard                      | LRFD |              |
| 6                    | 90         | 1.34                          | 1.98 | 48.2         |
|                      | 100        | 1.25                          | 1.98 | 58.1         |
|                      | 110        | 1.15                          | 1.98 | 71.8         |
|                      | 120        | 0.91                          | 1.98 | 116.4        |
|                      | 126        | -                             | 1.98 | -            |
|                      | 130        | 0.78                          | -    | -            |
|                      | 131        | 0.71                          | -    | -            |
| 8                    | 90         | 1.14                          | 1.98 | 73.5         |
|                      | 100        | 0.94                          | 1.98 | 109.7        |
|                      | 110        | 0.82                          | 1.98 | 142.8        |
|                      | 116        | -                             | 1.98 | -            |
|                      | 119        | 0.59                          | -    | -            |
| 8.67                 | 90         | 1.15                          | 1.98 | 71.6         |
|                      | 100        | 0.86                          | 1.98 | 130.4        |
|                      | 110        | 0.74                          | 1.98 | 168.4        |
|                      | 113        | -                             | 1.98 | -            |
|                      | 115        | 0.54                          | -    | -            |

#### 5.4.2.3 Initial Prestress Loss

The initial prestress loss is the combination of the losses due to elastic shortening and initial steel relaxation. The combined effect of the changes in these two losses was observed in the initial loss percent calculations between Standard and LRFD Specifications. The initial loss estimates provided by LRFD Specifications are found to be conservative as compared to the Standard Specifications.

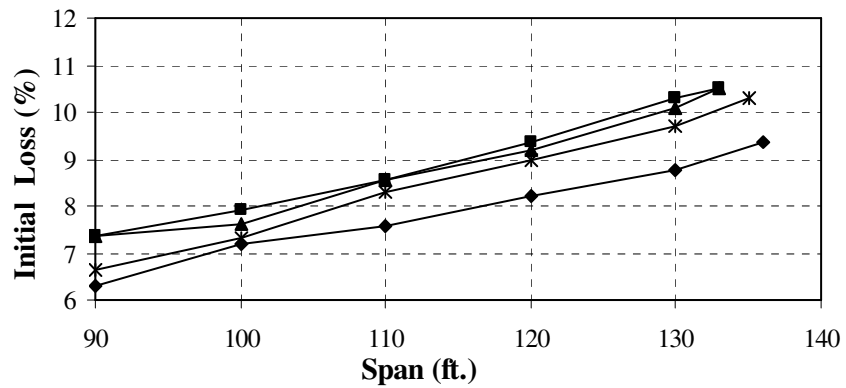
**Table 5.13. Comparison of Initial Prestress Loss (%) for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. Initial Loss (%) | Initial Loss Percent for LRFD |         |                |         |                |         |                |         |
|----------------------|------------|-----------------------|-------------------------------|---------|----------------|---------|----------------|---------|----------------|---------|
|                      |            |                       | Skew 0°                       |         | Skew 15°       |         | Skew 30°       |         | Skew 60°       |         |
|                      |            |                       | Init. Loss (%)                | Diff. % | Init. Loss (%) | Diff. % | Init. Loss (%) | Diff. % | Init. Loss (%) | Diff. % |
| 6                    | 90         | 6.31                  | 7.37                          | 16.7    | 7.37           | 16.7    | 7.37           | 16.7    | 6.65           | 5.4     |
|                      | 100        | 7.18                  | 7.92                          | 10.3    | 7.92           | 10.3    | 7.62           | 6.2     | 7.30           | 1.8     |
|                      | 110        | 7.57                  | 8.55                          | 12.9    | 8.55           | 12.9    | 8.55           | 12.9    | 8.31           | 9.7     |
|                      | 120        | 8.20                  | 9.36                          | 14.1    | 9.36           | 14.1    | 9.17           | 11.8    | 8.98           | 9.5     |
|                      | 130        | 8.75                  | 10.28                         | 17.5    | 10.28          | 17.5    | 10.08          | 15.3    | 9.71           | 11.0    |
|                      | 133        | -                     | 10.50                         | -       | 10.50          | -       | 10.50          | -       | -              | -       |
|                      | 135        | -                     | -                             | -       | -              | -       | -              | -       | 10.30          | -       |
|                      | 136        | 9.36                  | -                             | -       | -              | -       | -              | -       | -              | -       |
| 8                    | 90         | 7.53                  | 8.28                          | 9.9     | 8.28           | 9.9     | 8.29           | 10.1    | 7.69           | 2.2     |
|                      | 100        | 8.37                  | 9.16                          | 9.5     | 9.16           | 9.5     | 8.95           | 6.9     | 8.72           | 4.2     |
|                      | 110        | 8.93                  | 9.93                          | 11.3    | 9.93           | 11.3    | 9.76           | 9.4     | 9.59           | 7.4     |
|                      | 120        | 9.62                  | 10.91                         | 13.4    | 10.91          | 13.4    | 10.82          | 12.4    | 10.40          | 8.0     |
|                      | 124        | 10.07                 | -                             | -       | -              | -       | -              | -       | -              | -       |
|                      | 125        | -                     | -                             | -       | -              | -       | -              | -       | 10.91          | -       |
| 8.67                 | 90         | 7.78                  | 8.55                          | 9.9     | 8.55           | 9.9     | 8.56           | 10.0    | 7.99           | 2.7     |
|                      | 100        | 8.56                  | 9.38                          | 9.5     | 9.38           | 9.5     | 9.37           | 9.5     | 8.95           | 4.5     |
|                      | 110        | 9.22                  | 10.40                         | 12.8    | 10.40          | 12.8    | 10.26          | 11.3    | 9.76           | 5.9     |
|                      | 116        | -                     | 10.92                         | -       | 10.92          | -       | -              | -       | -              | -       |
|                      | 117        | -                     | -                             | -       | -              | -       | 10.90          | -       | -              | -       |
|                      | 119        | 10.07                 | -                             | -       | -              | -       | -              | -       | -              | -       |
|                      | 120        | -                     | -                             | -       | -              | -       | -              | -       | 10.82          | -       |
|                      | 121        | -                     | -                             | -       | -              | -       | -              | -       | 10.89          | -       |

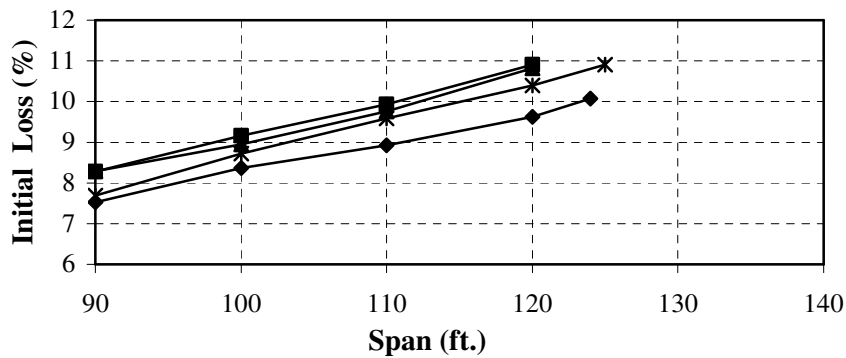
Except for few cases when skew angle was 60°, the increase in the initial loss percent was found to be in the range of 7 percent to 11 percent. The skew angle of 30° does not have a significant effect on initial loss percent however, the skew angle of 60° was found to decrease the initial loss percent significantly. This trend follows the trend of the loss due to elastic shortening as it is the major contributor to the initial losses.



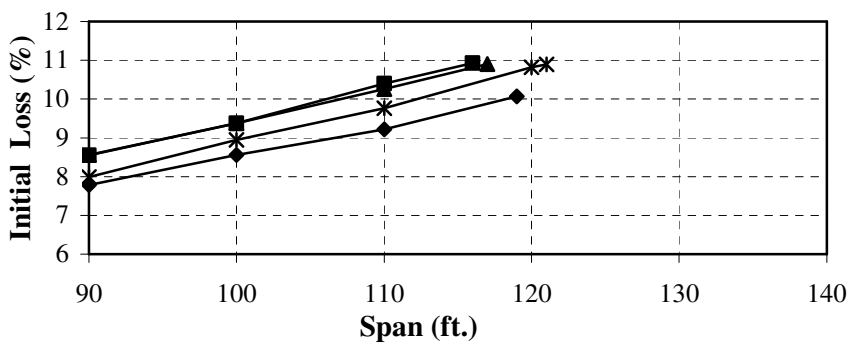
Table 5.13 presents the results for strand diameter of 0.5 in. Similar trends were observed for 0.6 in. diameter strands.



(a) Girder Spacing = 6 ft.



(b) Girder Spacing = 8 ft.



(c) Girder Spacing = 8.67 ft.



Figure 5.5. Comparison of Initial Prestress Loss (%) for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)

#### *5.4.2.4 Prestress Loss Due to Shrinkage of Concrete*

The Standard and LRFD Specifications prescribe the loss of prestress due to shrinkage of concrete as a function of relative humidity. For the relative humidity of 60 percent, loss due to shrinkage was found to be 8 ksi for both Standard and LRFD Specifications for all the cases.

#### *5.4.2.5 Total Prestress Loss Due to Steel Relaxation*

The total prestress loss due steel relaxation is the combination of loss due to initial relaxation and final relaxation of steel. The Standard Specifications specify empirical formulas to estimate the total loss due to steel relaxation half of which is considered to be at initial conditions and other half is considered in the final losses. This methodology is used by TxDOT Bridge Design Manual (TxDOT 2001). The LRFD Specifications specify an empirical formula to estimate the final prestress loss due to steel relaxation. The combined effect of the initial and final loss due to steel relaxation is presented in this section.

The estimate of total prestress loss due to steel relaxation provided by LRFD Specifications is found to be significantly conservative as compared to Standard Specifications. The conservatism is in the range of 78 percent to 135 percent for 6 ft., 94 percent to 182 percent for 8 ft. and 98 percent to 164 percent for 8.67 ft. girder spacing when 0.5 in. strands are used. The conservatism increases with the increase in span length and girder spacing. The increase in skew also increases the conservatism in the estimation of total relaxation losses. The results for 0.5 in. diameter strands are presented in Table 5.14. Similar trends were observed for 0.6 in. diameter strands.

**Table 5.14. Comparison of Total Relaxation Loss (CR<sub>s</sub>) for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. CR <sub>s</sub> (ksi) | LRFD                  |         |                       |         |                       |         |                       |         |
|----------------------|------------|----------------------------|-----------------------|---------|-----------------------|---------|-----------------------|---------|-----------------------|---------|
|                      |            |                            | Skew 0°               |         | Skew 15°              |         | Skew 30°              |         | Skew 60°              |         |
|                      |            |                            | CR <sub>s</sub> (ksi) | Diff. % | CR <sub>s</sub> (ksi) | Diff. % | CR <sub>s</sub> (ksi) | Diff. % | CR <sub>s</sub> (ksi) | Diff. % |
| 6                    | 90         | 2.91                       | 5.18                  | 78.0    | 5.18                  | 78.0    | 5.18                  | 78.0    | 5.46                  | 87.6    |
|                      | 100        | 2.51                       | 4.81                  | 91.2    | 4.81                  | 91.2    | 4.98                  | 98.3    | 5.17                  | 105.6   |
|                      | 110        | 2.34                       | 4.46                  | 90.7    | 4.46                  | 90.7    | 4.46                  | 90.8    | 4.62                  | 97.4    |
|                      | 120        | 1.97                       | 3.94                  | 99.6    | 3.94                  | 99.6    | 4.07                  | 106.2   | 4.20                  | 113.2   |
|                      | 130        | 1.60                       | 3.42                  | 113.8   | 3.42                  | 113.8   | 3.53                  | 120.5   | 3.76                  | 135.0   |
|                      | 133        | -                          | 3.28                  | -       | 3.28                  | -       | 3.28                  | -       | -                     | -       |
|                      | 135        | -                          | -                     | -       | -                     | -       | -                     | -       | 3.41                  | -       |
|                      | 136        | 1.29                       | -                     | -       | -                     | -       | -                     | -       | -                     | -       |
| 8                    | 90         | 2.45                       | 4.73                  | 93.1    | 4.73                  | 93.1    | 4.73                  | 93.0    | 5.09                  | 107.7   |
|                      | 100        | 2.00                       | 4.22                  | 111.0   | 4.22                  | 111.0   | 4.37                  | 118.5   | 4.53                  | 126.1   |
|                      | 110        | 1.67                       | 3.72                  | 123.0   | 3.72                  | 123.0   | 3.85                  | 130.6   | 3.98                  | 138.5   |
|                      | 120        | 1.22                       | 3.15                  | 157.4   | 3.15                  | 157.4   | 3.23                  | 163.7   | 3.45                  | 182.1   |
|                      | 124        | 0.97                       | -                     | -       | -                     | -       | -                     | -       | -                     | -       |
|                      | 125        | -                          | -                     | -       | -                     | -       | -                     | -       | 3.17                  | -       |
| 8.67                 | 90         | 2.32                       | 4.58                  | 97.3    | 4.58                  | 97.3    | 4.58                  | 97.1    | 4.93                  | 112.3   |
|                      | 100        | 1.90                       | 4.11                  | 115.9   | 4.11                  | 115.9   | 4.11                  | 115.9   | 4.41                  | 131.5   |
|                      | 110        | 1.48                       | 3.49                  | 135.5   | 3.49                  | 135.5   | 3.60                  | 143.1   | 3.88                  | 162.4   |
|                      | 116        | -                          | 3.15                  | -       | 3.15                  | -       | -                     | -       | -                     | -       |
|                      | 117        | -                          | -                     | -       | -                     | -       | 3.18                  | -       | -                     | -       |
|                      | 119        | 0.99                       | -                     | -       | -                     | -       | -                     | -       | -                     | -       |
|                      | 120        | -                          | -                     | -       | -                     | -       | -                     | -       | 3.27                  | -       |
|                      | 121        | -                          | -                     | -       | -                     | -       | -                     | -       | 3.22                  | -       |

#### 5.4.2.6 Prestress Loss Due to Creep of Concrete

The Standard and LRFD Specifications specify similar expressions for the estimation of prestress loss due to creep of concrete. The loss due to creep depends on the concrete stress at the center of gravity (c.g.) of prestressing strands due to dead loads before and after prestressing. Small difference was observed in the estimates of the loss due to concrete creep for Standard and LRFD Specifications. The estimate provided by



#### 5.4.2.7 Total Prestress Loss

The total loss of prestress is estimated based on Standard and LRFD Specifications. The combined effect of different losses results in total prestress loss estimates provided by LRFD Specifications that are slightly conservative as compared to those provided by Standard Specifications. The conservatism was found to be in the range of 7 percent to 16 percent for 6 ft., 1 percent to 12 percent for 8 ft. and 1 percent to 11 percent for 8.67 ft. girder spacing.

**Table 5.16. Comparison of Total Prestress Loss Percent for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**

[illegible]

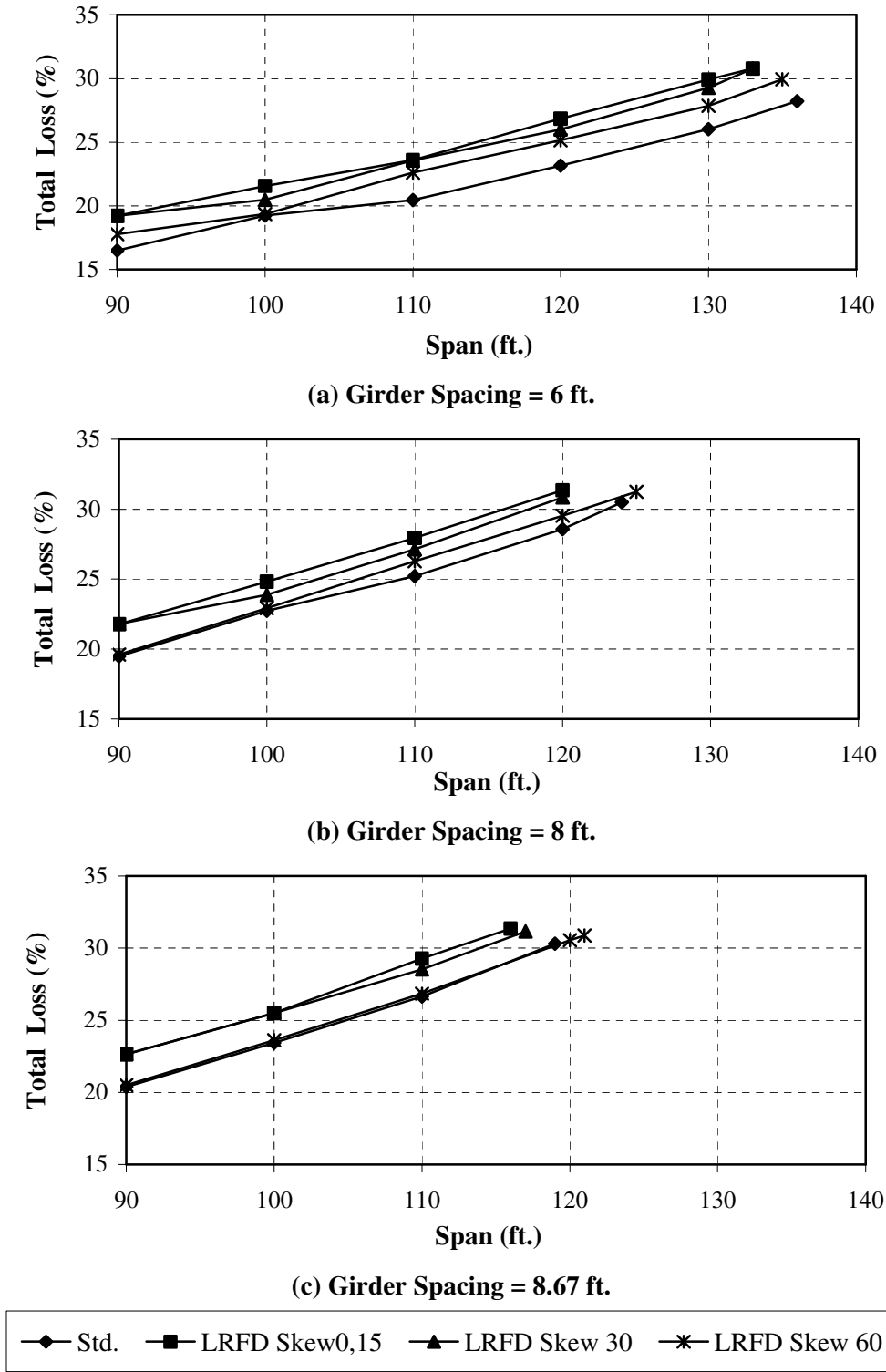


Figure 5.6. Comparison of Total Prestress Loss (%) for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)

The conservatism was found to be decreasing with the increase in span length, skew angle and girder spacing. The results for 0.5 in. diameter strands are presented in Table 5.15 and Figure 5.6.

#### **5.4.3. Impact on the Required Number of Prestressing Strands**

The number of strands required depends on the allowable stress limits and the stresses caused due to dead load and live load. There is a change in the allowable stress limits in LRFD Specifications and the live load stresses are also different. The Service III limit state that checks the bottom tensile stresses also impacts the prestressing strand requirements. The difference in the prestress losses is another factor that effects the final strand requirements. Strength limit state controls the number of strands only for one case when span length is 90 ft. with 6 ft. girder spacing.

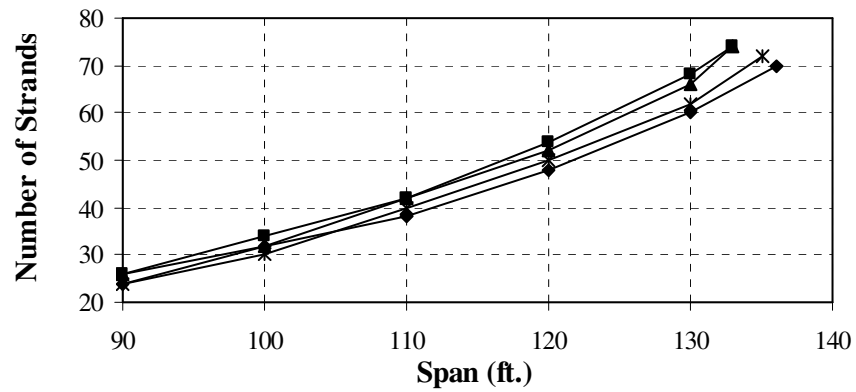
The LRFD Specifications require larger number of strands for most of the cases for 0.5 in diameter strands. The difference in the required number of strands by LRFD and Standard Specifications increases with the increase in span length and decreases with the increase in girder spacing and skew angle. For a few cases with skew angle of  $60^\circ$  the number of strands required by LRFD Specifications was found to be lesser than that of Standard Specifications. The difference was found to be in the range of -6 percent to 13 percent for 6 ft., -7 percent to 9 percent for 8 ft. and -7 percent to 7 percent for 8.67 ft. girder spacing. The results for 0.5 in diameter strands are presented in Table 5.16. Similar trends were found for 0.6 in. diameter strands. The results for 0.6 in. diameter strands are presented in Table 5.17. Figures 5.7 and 5.8 illustrate the comparison of strand requirement for Standard and LRFD specifications.

**Table 5.17. Comparison of Required Number of Strands for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**

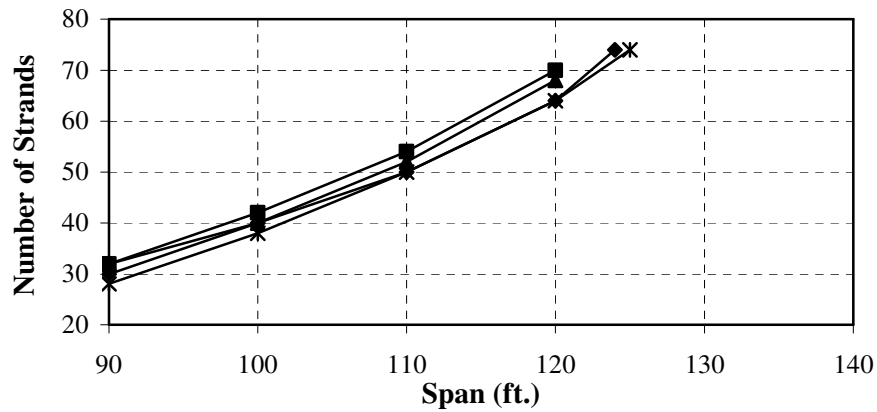
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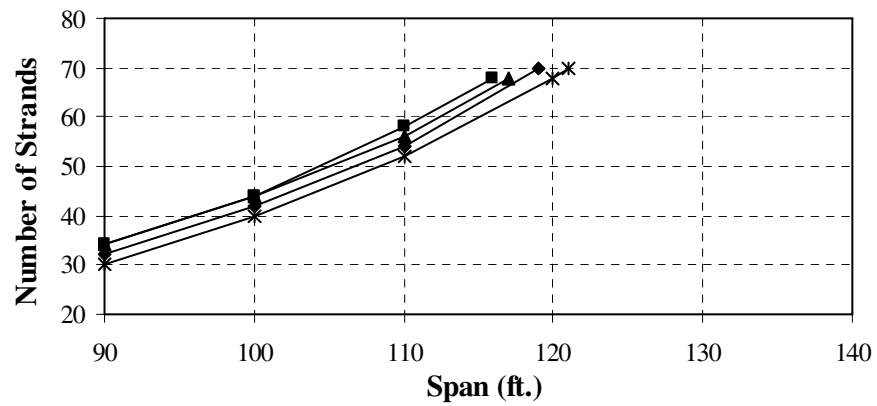




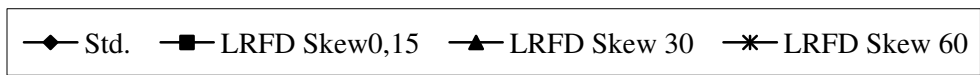
(a) Girder Spacing = 6 ft.



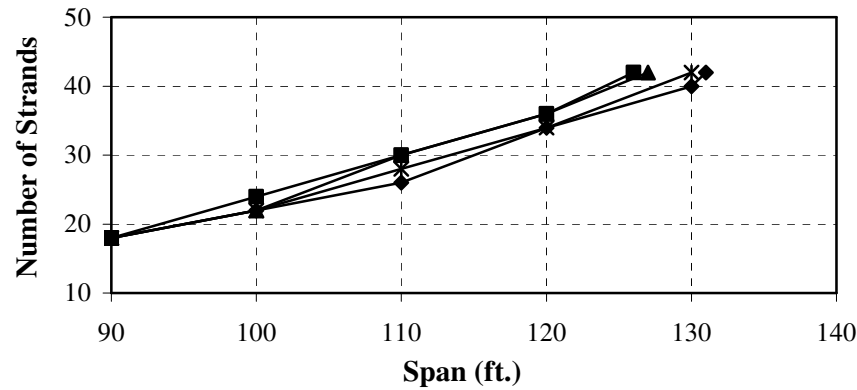
(b) Girder Spacing = 8 ft.



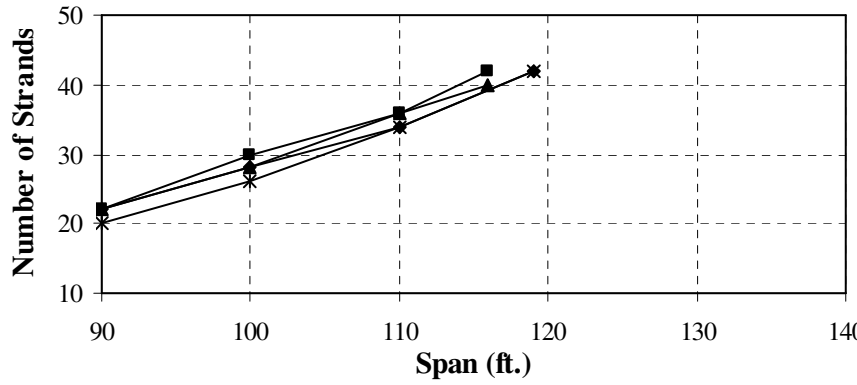
(c) Girder Spacing = 8.67 ft.



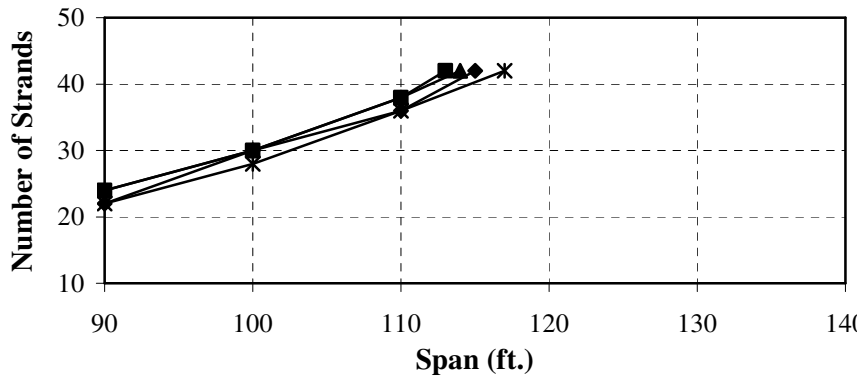
**Figure 5.7. Comparison of Required Number of Strands for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**



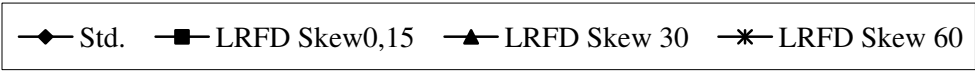
(a) Girder Spacing = 6 ft.



(b) Girder Spacing = 8 ft.



(c) Girder Spacing = 8.67 ft.



**Figure 5.8. Comparison of Required Number of Strands for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.6 in.)**

#### 5.4.4 Impact on Concrete Strengths

#### 5.4.4.1 Concrete Strength at Release

The optimized concrete strength at release depends on the stresses due to prestressing and the self weight of the girder. As there was a slight increase in the number of strands required for LRFD Specifications a subsequent increase in the required concrete strength at release was observed.

**Table 5.19. Comparison of Concrete Strength at Release ( $f'_{ci}$ ) for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**

[illegible]

The minimum strength at release was considered to be 4000 psi. For 90 ft. span length it was observed that minimum concrete strength governs. The LRFD Specifications yields a slightly conservative estimate of the concrete strength at release. The maximum difference was found to be 12 percent. The concrete strength at release is limited to 6750 psi. and in most of the cases it governs the maximum span length. The results for 0.5 in. diameter strands are presented in Table 5.19.

#### *5.4.4.2 Concrete Strength at Service*

The concrete strength at service is affected by the stresses at the midspan due to prestressing force, dead loads, superimposed loads and live loads. The concrete strength at service is limited to 8750 psi. However, this limitation does not affect the maximum span length as the initial concrete strength approaches its limits earlier than the final concrete strength. The minimum strength was considered as 5000 psi and for span lengths less than 110 ft. it was observed that this limit controls. Also the concrete strength at service cannot be smaller than the concrete strength at release. This limitation governs for a few cases for 0.5 in diameter strands and most of the cases for 0.6 in. diameter strands.

The LRFD Specifications do not have a significant effect on the concrete strength at service. A small reduction in the required concrete strength was observed for most of the cases, maximum difference being 9 percent. The results for 0.5 in. diameter strands are presented in Table 5.20.

**Table 5.20. Comparison of Concrete Strength at Service ( $f'_c$ ) for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | $f'_c$ (psi) | LRFD         |         |              |         |              |         |              |         |
|----------------------|------------|--------------|--------------|---------|--------------|---------|--------------|---------|--------------|---------|
|                      |            |              | Skew 0°      |         | Skew 15°     |         | Skew 30°     |         | Skew 60°     |         |
|                      |            |              | $f'_c$ (psi) | Diff. % | $f'_c$ (psi) | Diff. % | $f'_c$ (psi) | Diff. % | $f'_c$ (psi) | Diff. % |
| 6                    | 90         | 5000.0       | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
| 6                    | 100        | 5000.0       | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
| 6                    | 110        | 5000.0       | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
| 6                    | 120        | 5955.5       | 5930.6       | -0.4    | 5930.6       | -0.4    | 5692.1       | -4.4    | 5453.4       | -8.4    |
| 6                    | 130        | 7384.6       | 6833.0       | -7.5    | 6833.0       | -7.5    | 7215.8       | -2.3    | 6699.9       | -9.3    |
| 6                    | 133        | -            | 8619.5       | -       | 8619.5       | -       | 7683.3       | -       | -            | -       |
| 6                    | 135        | -            | -            | -       | -            | -       | -            | -       | 7937.6       | -       |
| 6                    | 136        | 8621.6       | -            | -       | -            | -       | -            | -       | -            | -       |
| 8                    | 90         | 5000.0       | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
| 8                    | 100        | 5000.0       | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
| 8                    | 110        | 5583.9       | 5893.2       | 5.5     | 5893.2       | 5.5     | 5655.3       | 1.3     | 5417.0       | -3.0    |
| 8                    | 120        | 7164.7       | 7598.9       | 6.1     | 7598.9       | 6.1     | 7639.9       | 6.6     | 6482.5       | -9.5    |
| 8                    | 124        | 8306.4       | -            | -       | -            | -       | -            | -       | -            | -       |
| 8                    | 125        | -            | -            | -       | -            | -       | -            | -       | 8305.0       | -       |
| 8.67                 | 90         | 5000.0       | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
| 8.67                 | 100        | 5000.0       | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
| 8.67                 | 110        | 5939.6       | 6057.8       | 2.0     | 6057.8       | 2.0     | 5837.0       | -1.7    | 5655.4       | -4.8    |
| 8.67                 | 116        | -            | 6780.5       | -       | 6780.5       | -       | -            | -       | -            | -       |
| 8.67                 | 117        | -            | -            | -       | -            | -       | 7261.9       | -       | -            | -       |
| 8.67                 | 119        | 7602.4       | -            | -       | -            | -       | -            | -       | -            | -       |
| 8.67                 | 120        | -            | -            | -       | -            | -       | -            | -       | 7222.7       | -       |
| 8.67                 | 121        | -            | -            | -       | -            | -       | -            | -       | 7806.4       | -       |

#### 5.4.5 Impact of AASHTO LRFD on Maximum Span Length

The maximum span lengths are limited by the concrete strength at release of 6750 psi and concrete strength at service of 8750 psi. The maximum span is not governed by maximum number of strands for any of the cases considered for the parametric study. The maximum allowable concrete strengths are reached when the strand number was in the range of 70 to 74, whereas AASHTO Type IV girder can hold up to 102 strands. Thus by relaxing the limit on concrete strengths longer spans can be

achieved. The LRFD Specifications have a reducing effect on the maximum span length. This is due to a slightly conservative estimate of the concrete strengths which reaches the limits earlier than the Standard Specifications. However, the difference between the maximum span lengths was found to be negligible. The difference was in the range of -4 percent to 2 percent for all the cases with strand diameter of 0.5 in. and 0.6 in. The results for maximum span length are presented in Table 5.21.

**Table 5.21. Comparison of Maximum Span Lengths for AASHTO Standard and LRFD Specifications**

| Strand Dia. (in.) | Girder Spacing (ft.) | Std. Max. Span (ft.) | LRFD            |           |                 |           |                 |           |                 |           |
|-------------------|----------------------|----------------------|-----------------|-----------|-----------------|-----------|-----------------|-----------|-----------------|-----------|
|                   |                      |                      | Skew 0°         |           | Skew 15°        |           | Skew 30°        |           | Skew 60°        |           |
|                   |                      |                      | Max. Span (ft.) | Diff. (%) | Max. Span (ft.) | Diff. (%) | Max. Span (ft.) | Diff. (%) | Max. Span (ft.) | Diff. (%) |
| 0.5               | 6                    | 136                  | 133             | -2.2      | 133             | -2.2      | 133             | -2.2      | 135             | -0.7      |
|                   | 8                    | 124                  | 120             | -3.2      | 120             | -3.2      | 120             | -3.2      | 125             | 0.8       |
|                   | 8.67                 | 119                  | 116             | -2.5      | 116             | -2.5      | 117             | -1.7      | 121             | 1.7       |
| 0.6               | 6                    | 131                  | 126             | -3.8      | 126             | -3.8      | 127             | -3.1      | 130             | -0.8      |
|                   | 8                    | 119                  | 116             | -2.5      | 116             | -2.5      | 116             | -2.5      | 119             | 0.0       |
|                   | 8.67                 | 115                  | 113             | -1.7      | 113             | -1.7      | 114             | -0.9      | 117             | 1.7       |

## **5.5 IMPACT OF AASHTO LRFD SPECIFICATIONS ON FLEXURAL STRENGTH LIMIT STATE**

### **5.5.1 General**

The impact of LRFD Specifications on the requirements of ultimate flexural strength limit state design is discussed in following section. The change in the load combination, the required concrete strengths and the number of strands from service limit state effects the ultimate flexural strength limit. The impact of change in the definition of the rectangular section behavior and flanged section behavior in LRFD Specifications is discussed. The reinforcement limits have also been changed in the LRFD specifications, however for all the cases of Standard and LRFD Specifications the sections are found to be under reinforced.

### **5.5.2 Impact on Design Moment**

The load combinations for ultimate limit state are significantly changed from Standard to LRFD Specifications. The load factors for moments due to live load and dead loads except wearing surface load specified by LRFD Specifications are smaller than the Standard Specifications. The load factor for moment due to wearing surface load is increased in the LRFD Specifications. The live load moments specified by LRFD specifications are larger than that of Standard Specifications. The combined effect of these two changes results in the design moments that are comparable.

The LRFD Specifications yields design moments that are in general slightly conservative as compared to the Standard Specifications. The conservatism is found to decrease with the increase in span length, girder spacing and skew angle beyond 30°. The design moments for skew angle of 60° are less conservative as compared to the Standard Specifications. The difference in the design moments was found to be in the range of -2 percent to 8 percent for 6 ft., -8 percent to 4 percent for 8 ft. and -10 percent to 3 percent for 8.67 ft. girder spacing. The strand diameter does not have any effect on



the design moments. The comparison of the design moments specified by Standard and LRFD specifications is presented in Table 5.22.

**Table 5.22. Comparison of Factored Ultimate Moment ( $M_u$ ) for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. $M_u$ (k-ft.) | LRFD          |         |               |         |               |         |               |         |
|----------------------|------------|--------------------|---------------|---------|---------------|---------|---------------|---------|---------------|---------|
|                      |            |                    | Skew 0°       |         | Skew 15°      |         | Skew 30°      |         | Skew 60°      |         |
|                      |            |                    | $M_u$ (k-ft.) | Diff. % | $M_u$ (k-ft.) | Diff. % | $M_u$ (k-ft.) | Diff. % | $M_u$ (k-ft.) | Diff. % |
| 6                    | 90         | 3960.7             | 4278.7        | 8.0     | 4278.7        | 8.0     | 4200.0        | 6.0     | 3869.9        | -2.3    |
|                      | 100        | 4690.2             | 5053.6        | 7.7     | 5053.6        | 7.7     | 4971.5        | 6.0     | 4626.8        | -1.4    |
|                      | 110        | 5470.3             | 5884.6        | 7.6     | 5884.6        | 7.6     | 5799.2        | 6.0     | 5440.7        | -0.5    |
|                      | 120        | 6299.9             | 6771.2        | 7.5     | 6771.2        | 7.5     | 6682.6        | 6.1     | 6310.9        | 0.2     |
|                      | 130        | 7179.5             | 7713.7        | 7.4     | 7713.7        | 7.4     | 7622.1        | 6.2     | 7237.7        | 0.8     |
|                      | 133        | -                  | 8007.3        | -       | 8007.3        | -       | 7914.9        | -       | -             | -       |
|                      | 135        | -                  | -             | -       | -             | -       | -             | -       | 7722.4        | -       |
|                      | 136        | 7731.2             | -             | -       | -             | -       | -             | -       | -             | -       |
| 8                    | 90         | 4932.0             | 5117.6        | 3.8     | 5117.6        | 3.8     | 5006.4        | 1.5     | 4540.0        | -7.9    |
|                      | 100        | 5821.5             | 6035.1        | 3.7     | 6035.1        | 3.7     | 5919.1        | 1.7     | 5432.5        | -6.7    |
|                      | 110        | 6769.4             | 7017.9        | 3.7     | 7017.9        | 3.7     | 6897.3        | 1.9     | 6391.5        | -5.6    |
|                      | 120        | 7774.6             | 8065.2        | 3.7     | 8065.2        | 3.7     | 7940.3        | 2.1     | 7416.3        | -4.6    |
|                      | 124        | 8192.8             | -             | -       | -             | -       | -             | -       | -             | -       |
|                      | 125        | -                  | -             | -       | -             | -       | -             | -       | 7953.5        | -       |
| 8.67                 | 90         | 5232.1             | 5365.6        | 2.6     | 5365.6        | 2.6     | 5243.0        | 0.2     | 4728.9        | -9.6    |
|                      | 100        | 6169.1             | 6323.7        | 2.5     | 6323.7        | 2.5     | 6195.8        | 0.4     | 5659.4        | -8.3    |
|                      | 110        | 7166.6             | 7349.4        | 2.6     | 7349.4        | 2.6     | 7216.6        | 0.7     | 6659.1        | -7.1    |
|                      | 116        | -                  | 7996.9        | -       | 7996.9        | -       | -             | -       | -             | -       |
|                      | 117        | -                  | -             | -       | -             | -       | 7971.0        | -       | -             | -       |
|                      | 119        | 8114.9             | -             | -       | -             | -       | -             | -       | -             | -       |
|                      | 120        | -                  | -             | -       | -             | -       | -             | -       | 7727.0        | -       |
|                      | 121        | -                  | -             | -       | -             | -       | -             | -       | 7837.6        | -       |

### 5.5.3 Impact on Section Behavior

The impact of LRFD Specifications on the section behavior is discussed in this section. The Standard Specifications defines a section to be behaving as a rectangular

section if the depth of equivalent stress block is less than the thickness of compression flange (slab). The LRFD Specifications use the location of neutral axis to categorize the section behavior as rectangular or flanged. The section is defined to be rectangular if the neutral axis lies in the compression flange (slab). The expression specified by LRFD Specifications for the determination of depth of neutral axis is different from the Standard Specifications. The location of the stress block and neutral axis for Standard and LRFD Specifications is presented in Figures 5.- and 5.-.

The flanged section behavior is categorized into two cases for Standard Specifications. The first case when the depth of stress block is less than the sum of the slab and girder flange thickness and the second case when the depth of stress block exceeds the sum of the thickness of slab and girder flange. It was observed that for Standard Specifications most of the sections have rectangular section behavior and for few cases when the span length is larger than 120 ft. the stress block enters the girder flange. The stress block does not enter the web portion of the girder for any of the cases considered in the parametric study.

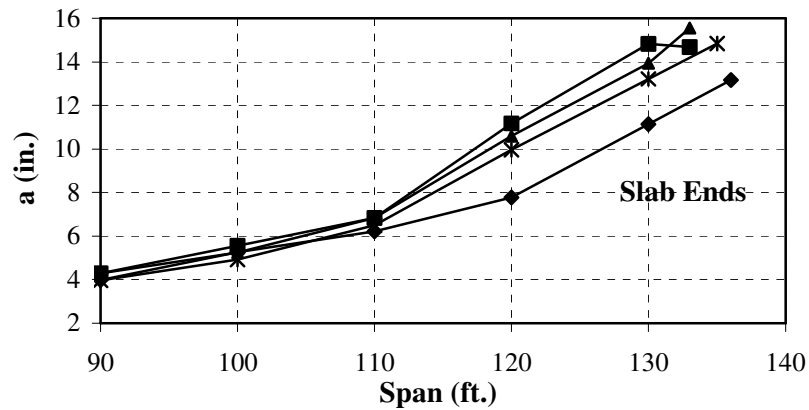
The flanged section behavior is divided into three categories for LRFD specifications. The first case when the neutral axis lies in the flange of the girder, the second case when the neutral axis lies in the fillet portion of the girder and the third case when the neutral axis lies in the web of the girder. It was observed that for span lengths up to 110 ft. the section behaves as a rectangular section for most cases. For span lengths up to 120 ft. the neutral axis lies in the girder flange and thereafter in the fillet portion of the girder. The neutral axis does not lie in the girder web for any of the cases considered for this study.

**Table 5.23. Section Behavior for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**

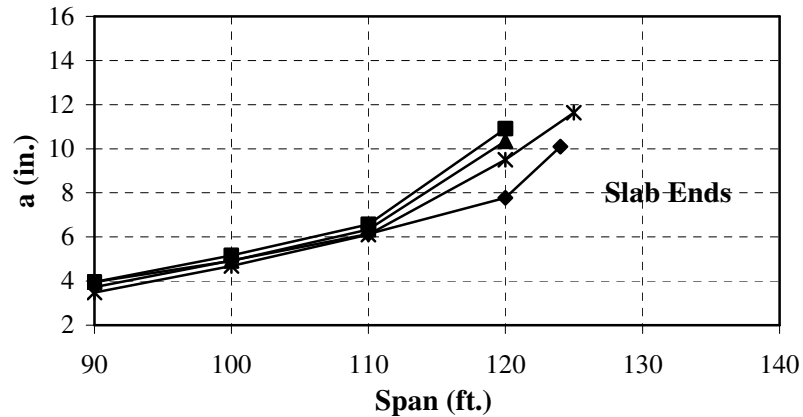
| Girder Spacing (ft.) | Span (ft.) | Standard Section Behavior | LRFD Section Behavior |                       |                       |                       |
|----------------------|------------|---------------------------|-----------------------|-----------------------|-----------------------|-----------------------|
|                      |            |                           | Skew 0°               | Skew 15°              | Skew 30°              | Skew 60°              |
| 6                    | 90         | Rec.                      | Rec.                  | Rec.                  | Rec.                  | Rec.                  |
|                      | 100        | Rec.                      | Rec.                  | Rec.                  | Rec.                  | Rec.                  |
|                      | 110        | Rec.                      | Flanged <sup>*</sup>  | Flanged <sup>*</sup>  | Flanged <sup>*</sup>  | Rec.                  |
|                      | 120        | Rec.                      | Flanged <sup>*</sup>  | Flanged <sup>*</sup>  | Flanged <sup>*</sup>  | Flanged <sup>*</sup>  |
|                      | 130        | Flanged <sup>*</sup>      | Flanged <sup>**</sup> | Flanged <sup>**</sup> | Flanged <sup>**</sup> | Flanged <sup>**</sup> |
|                      | 133        | -                         | Flanged <sup>**</sup> | Flanged <sup>**</sup> | Flanged <sup>**</sup> | -                     |
|                      | 135        | -                         | -                     | -                     | -                     | Flanged <sup>**</sup> |
|                      | 136        | Flanged <sup>*</sup>      | -                     | -                     | -                     | -                     |
| 8                    | 90         | Rec.                      | Rec.                  | Rec.                  | Rec.                  | Rec.                  |
|                      | 100        | Rec.                      | Rec.                  | Rec.                  | Rec.                  | Rec.                  |
|                      | 110        | Rec.                      | Rec.                  | Rec.                  | Rec.                  | Rec.                  |
|                      | 120        | Rec.                      | Flanged <sup>*</sup>  | Flanged <sup>*</sup>  | Flanged <sup>*</sup>  | Flanged <sup>*</sup>  |
|                      | 124        | Flanged <sup>*</sup>      | -                     | -                     | -                     | -                     |
|                      | 125        | -                         | -                     | -                     | -                     | Flanged <sup>*</sup>  |
| 8.67                 | 90         | Rec.                      | Rec.                  | Rec.                  | Rec.                  | Rec.                  |
|                      | 100        | Rec.                      | Rec.                  | Rec.                  | Rec.                  | Rec.                  |
|                      | 110        | Rec.                      | Rec.                  | Rec.                  | Rec.                  | Rec.                  |
|                      | 116        | -                         | Flanged <sup>*</sup>  | Flanged <sup>*</sup>  | -                     | -                     |
|                      | 117        | -                         | -                     | -                     | Flanged <sup>*</sup>  | -                     |
|                      | 119        | Rec.                      | -                     | -                     | -                     | -                     |
|                      | 120        | -                         | -                     | -                     | -                     | Flanged <sup>*</sup>  |
|                      | 121        | -                         | -                     | -                     | -                     | Flanged <sup>*</sup>  |

Notes:

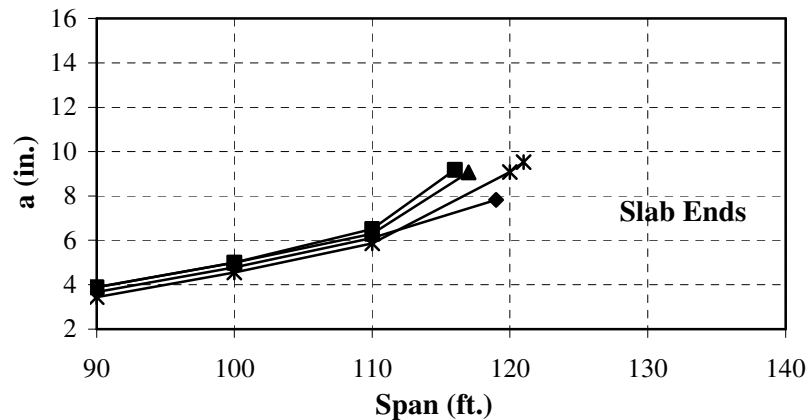
- 1) Flanged<sup>\*</sup> : The section behaves as a flanged section with neutral axis lying in the girder flange for LRFD Specifications and stress block lying in the girder flange for Standard Specifications.
- 2) Flanged<sup>\*\*</sup> : The section behaves as a flanged section with neutral axis lying in the fillet area of the girder for LRFD Specifications and stress block lying in the fillet area of the girder for Standard Specifications.



(a) Girder Spacing = 6 ft.



(b) Girder Spacing = 8 ft.



(c) Girder Spacing = 8.67 ft.

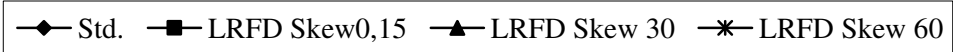
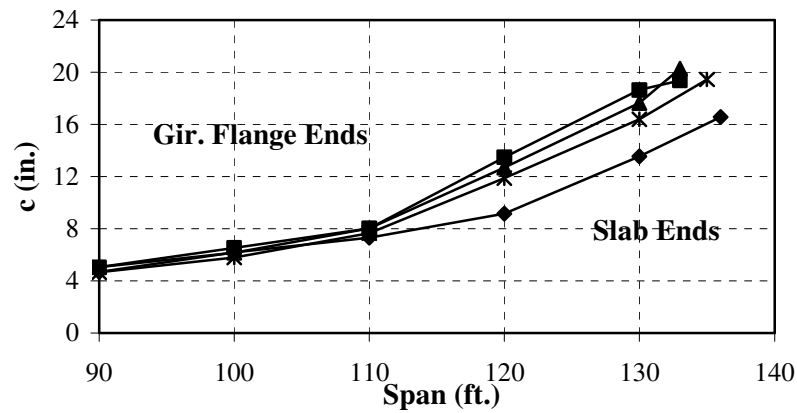
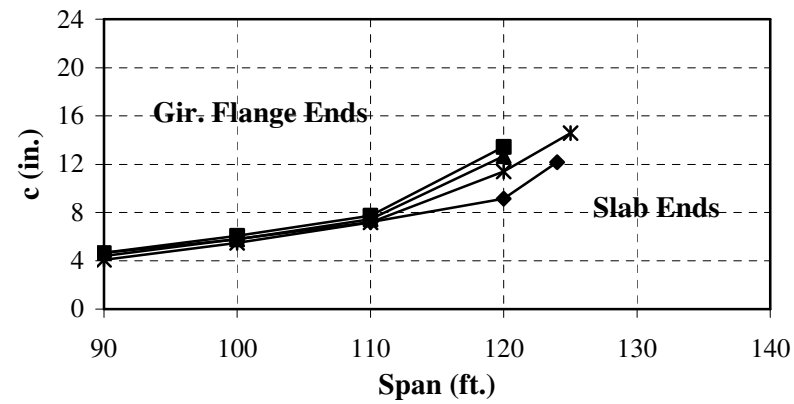


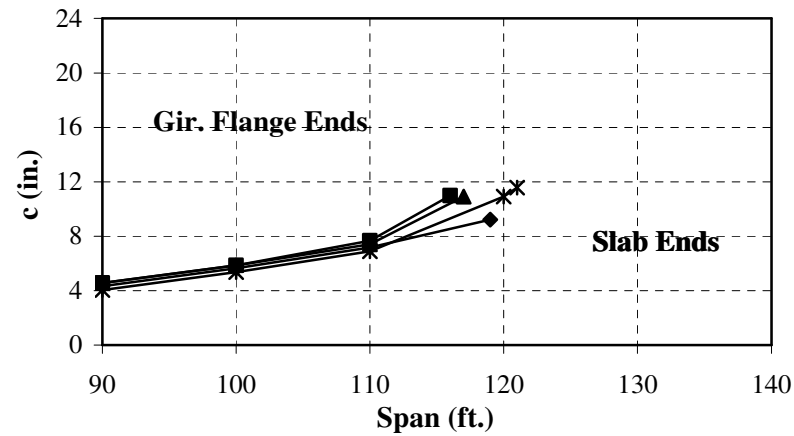
Figure 5.9. Comparison of depth of equivalent stress block (in.) for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)



(a) Girder Spacing = 6 ft.



(b) Girder Spacing = 8 ft.



(c) Girder Spacing = 8.67 ft.

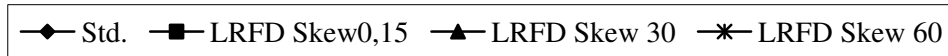


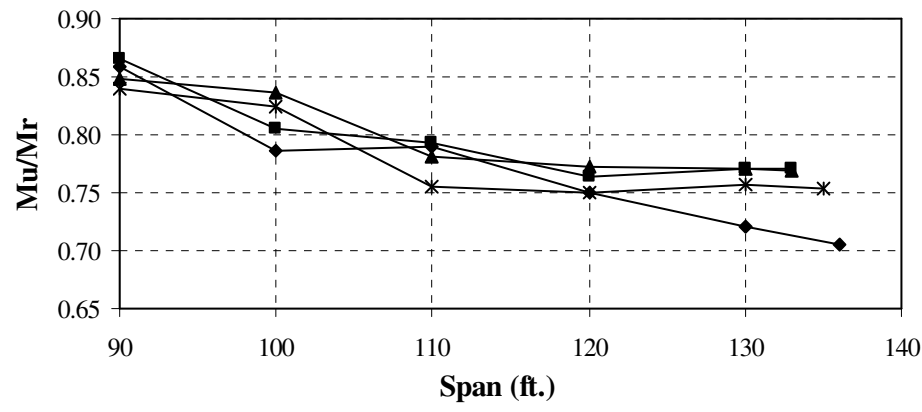
Figure 5.10. Comparison of depth of Neutral Axis (in.) for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)



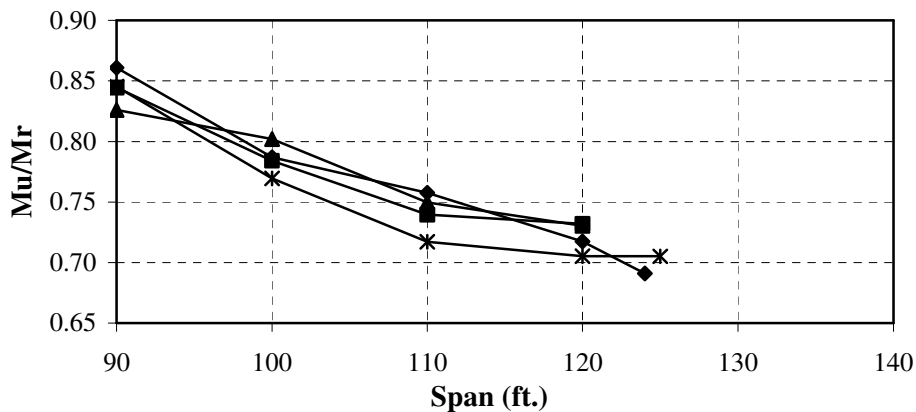
The moment resistance specified by LRFD Specifications was found to be slightly conservative as compared to the Standard specifications for most of the cases with 6 ft. girder spacing. For other girder spacing, the moment resistance specified by LRFD Specifications is less conservative as compared to the Standard Specifications. The conservatism was found to be increasing with increase in span length and decreasing with the increase in girder spacing and skew angle. For skew angle of 60° the moment resistance was found to be less conservative as compared to the Standard Specifications.

**Table 5.25. Comparison of  $M_u/M_r$  ratio for AASHTO Standard and LRFD Specifications (Strand Dia. = 0.5 in.)**

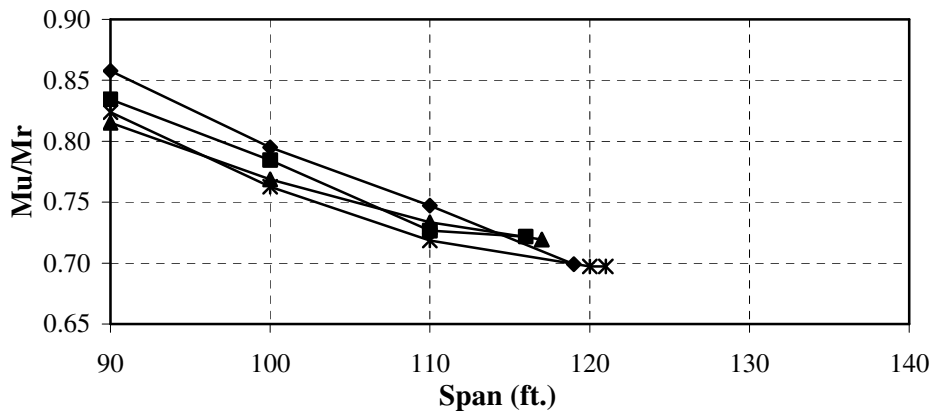
[illegible]



(a) Girder Spacing = 6 ft.



(b) Girder Spacing = 8 ft.



(c) Girder Spacing = 8.67 ft.

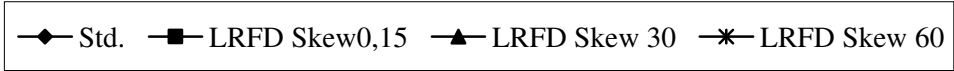


Figure 5.11. Comparison of  $M_u/M_r$  ratio for Standard and LRFD Specifications (Strand Dia. = 0.5 in.)



The difference between the moment resistance capacities predicted by Standard and LRFD Specifications is found to be in the range of -4 percent to 7 percent for 6 ft., -5 percent to 2 percent for 8 ft. and -4 percent to -3 percent for 8.67 ft. girder spacing. The comparison of the moment capacities is presented in Table 5.24. The impact of LRFD specifications on the  $M_u/M_r$  ratio is also investigated. This ratio signifies the level of safety at the ultimate conditions. A well defined trend was not observed for this ratio, however very small difference was observed between the  $M_u/M_r$  ratios for two specifications. The comparison is presented in Table 5.25 and the results are illustrated in Figure 5.11.

## **5.6 IMPACT OF LRFD SPECIFICATIONS ON CAMBER**

The camber was calculated using the Hyperbolic Functions method proposed by Sinno et al. (1968). This method is used by TxDOT for the evaluation of camber. As the camber is evaluated using the same methodology for both the specifications, a small difference between the cambers is observed. The results for the camber are summarized in Table 5.26. The cambers for LRFD designs were larger as compared to those for standard designs. This increase is due to larger required concrete strength at release, which increases the modulus of elasticity of the concrete at release. The maximum difference in the camber is 21 percent for 6 ft. girder spacing, 13 percent for 8 ft. girder spacing and 11 percent for 8.67 ft. girder spacing.

**Table 5.26. Comparison of Camber (Strand Dia. = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. Camber (ft.) | LRFD         |         |              |         |              |         |              |         |
|----------------------|------------|-------------------|--------------|---------|--------------|---------|--------------|---------|--------------|---------|
|                      |            |                   | Skew 0°      |         | Skew 15°     |         | Skew 30°     |         | Skew 60°     |         |
|                      |            |                   | Camber (ft.) | Diff. % | Camber (ft.) | Diff. % | Camber (ft.) | Diff. % | Camber (ft.) | Diff. % |
| 6                    | 90         | 0.12              | 0.12         | 3.1     | 0.12         | 3.1     | 0.12         | 3.1     | 0.11         | -1.3    |
|                      | 100        | 0.17              | 0.19         | 11.5    | 0.19         | 11.5    | 0.17         | -1.6    | 0.15         | 15.1    |
|                      | 110        | 0.21              | 0.25         | 20.3    | 0.25         | 20.3    | 0.25         | 20.3    | 0.23         | 9.3     |
|                      | 120        | 0.28              | 0.34         | 21.1    | 0.34         | 21.1    | 0.32         | 14.0    | 0.30         | 6.8     |
|                      | 130        | 0.36              | 0.36         | 2.3     | 0.36         | 2.3     | 0.36         | 2.2     | 0.35         | -1.6    |
|                      | 133        | -                 | 0.33         | -       | 0.33         | -       | 0.34         | -       | -            | -       |
|                      | 135        | -                 | -            | -       | -            | -       | -            | -       | 0.33         | -       |
|                      | 136        | 0.34              | -            | -       | -            | -       | -            | -       | -            | -       |
| 8                    | 90         | 0.16              | 0.18         | 10.0    | 0.18         | 10.0    | 0.18         | 10.0    | 0.14         | -       |
|                      | 100        | 0.24              | 0.26         | 5.9     | 0.26         | 5.9     | 0.24         | -1.8    | 0.22         | -9.6    |
|                      | 110        | 0.32              | 0.35         | 8.3     | 0.35         | 8.3     | 0.33         | 3.4     | 0.32         | -1.7    |
|                      | 120        | 0.40              | 0.38         | -4.1    | 0.38         | -4.1    | 0.38         | -3.8    | 0.39         | -1.3    |
|                      | 124        | 0.37              | -            | -       | -            | -       | -            | -       | -            | -       |
|                      | 125        | -                 | -            | -       | -            | -       | -            | -       | 0.36         | -       |
| 8.67                 | 90         | 0.18              | 0.19         | 8.5     | 0.19         | 8.5     | 0.19         | 8.5     | 0.16         | -       |
|                      | 100        | 0.26              | 0.28         | 5.1     | 0.28         | 5.1     | 0.28         | 5.1     | 0.24         | -8.9    |
|                      | 110        | 0.36              | 0.36         | 2.0     | 0.36         | 2.0     | 0.35         | -1.6    | 0.33         | -6.4    |
|                      | 116        | -                 | 0.39         | -       | 0.39         | -       | -            | -       | -            | -       |
|                      | 117        | -                 | -            | -       | -            | -       | 0.38         | -       | -            | -       |
|                      | 119        | 0.39              | -            | -       | -            | -       | -            | -       | -            | -       |
|                      | 120        | -                 | -            | -       | -            | -       | -            | -       | 0.38         | -       |
|                      | 121        | -                 | -            | -       | -            | -       | -            | -       | 0.38         | -       |

## 5.7 IMPACT OF AASHTO LRFD ON SHEAR DESIGN

The LRFD Specifications employs a different methodology for the transverse shear design as compared to the Standard Specifications. This difference in the methodology impacted the transverse shear design significantly. The transverse shear reinforcement area was found to be increasing significantly as compared to the Standard

Specifications. The interface shear reinforcement area was found to be increasing significantly for the LRFD Specifications. These changes in the transverse and interface shear reinforcements are addressed in Section 7. The results and comparison is also presented in Section 7.

## 6. RESULTS FOR TYPE C GIRDERS

### 6.1 INTRODUCTION

A parametric study was conducted for Type C prestressed concrete bridge girders. Several cases were considered based on the parameters summarized in Table 6.1. The procedure outlined in Section 4 was employed to evaluate the impact of the AASHTO LRFD Specifications on the design of Type C bridge girders. The results obtained from the design program for designs based on both the Standard and LRFD Specifications were validated using TxDOT's PRSTRS14 (TxDOT 2004) bridge design software. TxDOT's procedures were used for optimizing the number of strands and concrete strengths. This section provides a summary of results of the parametric study for Type C bridge girders. The impact of LRFD specifications on various design results is discussed.

**Table 6.1. Design Parameters for Type C Girder**

| Parameter                               | Description / Selected Values   |
|---|---|
| Design Codes                            | AASHTO Standard Specifications, 17th Edition (2002)<br>AASHTO LRFD Specifications, 3rd Edition (2004)   |
| Girder Spacing (ft.)                    | 6'-0", 8'-0" and 8'-8"  |
| Spans                                   | 40 ft. to maximum span at 10 ft. intervals  |
| Strand Diameter (in.)                   | 0.5 and 0.6   |
| Concrete Strength at Release, $f'_{ci}$ | Varied from 4000 to 6750 psi for design with optimum number of strands  |
| Concrete Strength at Service, $f'_c$    | Varied from 5000 to 8500 psi for design with optimum number of strands ( $f'_c$ may be increased up to 8750 psi for optimization on longer spans) |
| Skew Angle                              | 0°, 15°, 30° and 60°  |

## **6.2 IMPACT OF AASHTO LRFD SPECIFICATIONS**

The requirements for service load limit state design, flexural strength limit state design, transverse shear design, and interface shear design are evaluated in the parametric study. The designs based on LRFD Specifications are found to be conservative, in general as compared to the designs based on Standard Specifications. This conservatism is caused due to the increase in live load moments, more restrictive limits for service load design, and difference in the shear design approach. The effect of LRFD Specifications on the maximum allowable span length was investigated. The effect was found to be small.

The following sections provide the summary of differences observed in the designs based on Standard and LRFD Specifications. This includes the differences occurring in the undistributed and distributed live load moments, the distribution factors, the number of strands required, and required concrete strengths at release and at service. The differences observed in the flexural strength limit state design are provided in the following sections. The effect on camber is also evaluated and summarized. The differences in the transverse shear design and interface shear design are provided in Section 7.

## **6.3 IMPACT OF LRFD SPECIFICATIONS ON LIVE LOAD MOMENTS AND SHEARS**

### **6.3.1 General**

The Standard Specifications specify the live load to be taken as an HS-20 truck load, tandem load, or lane load, whichever produces the maximum effect at the section considered. The LRFD Specifications specifies a different live load model HL-93, which is a combination of the HS-20 truck and lane load, or tandem load and lane load, whichever produces maximum effect at the section of interest. The live load governing the moments and shears at the sections of interest for the cases considered in the parametric study was determined and are summarized below. The undistributed live load

moments at midspan and shears at critical section were calculated for each case and the representative differences are presented in this section.

There is a significant difference in the formulas for the distribution and impact factors specified by the Standard and the LRFD Specifications. The impact factors are applicable to truck, lane, and tandem loadings for designs based on Standard Specifications, whereas the LRFD Specifications does not require the lane load to be increased for the impact loading. The effect of the LRFD Specifications on the distribution and impact factors is evaluated and the results are summarized. The combined effect of the undistributed moments and shears and the distribution and impact factors on the distributed live load moments and shears was observed. The differences observed in the distributed live load moments at midspan and shears at the critical sections are presented below.

### **6.3.2 Governing Live Load for Moments and Shears**

The live load producing the maximum moment at mid-span and maximum shears at the critical section for shear is investigated. The critical section for shear in the designs based on Standard Specifications is taken as  $h/2$ , where  $h$  is the depth of composite section. For designs based on LRFD Specifications the critical section is calculated using an iterative process specified by the specifications. The governing live loads are summarized in the Tables 6.2 and 6.3.

**Table 6.2. Governing Live Load Moments at Midspan and Shears at  
Critical Section for Standard Specifications for Type C Girder**

| <b>Strand<br/>Diameter (in.)</b> | <b>Girder<br/>Spacing (ft.)</b> | <b>Span<br/>(ft.)</b> | <b>Governing Live Load<br/>for Moment</b> | <b>Governing Live Load<br/>for Shear</b> |
|----------------------------------|---------------------------------|-----------------------|---|--|
| 0.5                              | 6                               | 40                    | Truck Loading                             | Truck Loading                            |
|                                  |                                 | 50                    |   |  |
|                                  |                                 | 60                    |   |  |
|                                  |                                 | 70                    |   |  |
|                                  |                                 | 80                    |   |  |
|                                  |                                 | 90                    |   |  |
|                                  |                                 | 96                    |   |  |
|                                  | 8                               | 40                    | Truck Loading                             | Truck Loading                            |
|                                  |                                 | 50                    |   |  |
|                                  |                                 | 60                    |   |  |
|                                  |                                 | 70                    |   |  |
|                                  |                                 | 80                    |   |  |
|                                  |                                 | 83                    |   |  |
|                                  | 8.67                            | 40                    | Truck Loading                             | Truck Loading                            |
|                                  |                                 | 50                    |   |  |
|                                  |                                 | 60                    |   |  |
|                                  |                                 | 70                    |   |  |
|                                  |                                 | 80                    |   |  |
| 0.6                              | 6                               | 40                    | Truck Loading                             | Truck Loading                            |
|                                  |                                 | 50                    |   |  |
|                                  |                                 | 60                    |   |  |
|                                  |                                 | 70                    |   |  |
|                                  |                                 | 80                    |   |  |
|                                  |                                 | 90                    |   |  |
|                                  |                                 | 95                    |   |  |
|                                  | 8                               | 40                    | Truck Loading                             | Truck Loading                            |
|                                  |                                 | 50                    |   |  |
|                                  |                                 | 60                    |   |  |
|                                  |                                 | 70                    |   |  |
|                                  |                                 | 80                    |   |  |
|                                  |                                 | 82                    |   |  |
|                                  | 8.67                            | 40                    | Truck Loading                             | Truck Loading                            |
|                                  |                                 | 50                    |   |  |
|                                  |                                 | 60                    |   |  |
|                                  |                                 | 70                    |   |  |
|                                  |                                 | 79                    |   |  |

**Table 6.3. Governing Live Load Moments at Midspan and Shears at Critical Section for LRFD Specifications for Type C Girder (Skew = 0°)**

| Strand Diameter (in.) | Girder Spacing (ft.) | Span (ft.) | Governing Live Load for Moment | Governing Live Load for Shear |
|-----------------------|----------------------|------------|--------------------------------|-------------------------------|
| 0.5                   | 6                    | 40         | Tandem+Lane Loading            | Truck+Lane Loading            |
|                       |                      | 50         | Truck+Lane Loading             |                               |
|                       |                      | 60         |                                |                               |
|                       |                      | 70         |                                |                               |
|                       |                      | 80         |                                |                               |
|                       |                      | 90         |                                |                               |
|                       |                      | 95         |                                |                               |
|                       | 8                    | 40         | Tandem+Lane Loading            | Truck+Lane Loading            |
|                       |                      | 50         | Truck+Lane Loading             |                               |
|                       |                      | 60         |                                |                               |
|                       |                      | 70         |                                |                               |
|                       |                      | 80         |                                |                               |
|                       |                      | 83         |                                |                               |
|                       | 8.67                 | 40         | Tandem+Lane Loading            | Truck+Lane Loading            |
|                       |                      | 50         | Truck+Lane Loading             |                               |
|                       |                      | 60         |                                |                               |
|                       |                      | 70         |                                |                               |
|                       |                      | 80         |                                |                               |
| 0.6                   | 6                    | 40         | Tandem+Lane Loading            | Truck+Lane Loading            |
|                       |                      | 50         | Truck+Lane Loading             |                               |
|                       |                      | 60         |                                |                               |
|                       |                      | 70         |                                |                               |
|                       |                      | 80         |                                |                               |
|                       |                      | 90         |                                |                               |
|                       |                      | 92         |                                |                               |
|                       | 8                    | 40         | Tandem+Lane Loading            | Truck+Lane Loading            |
|                       |                      | 50         | Truck+Lane Loading             |                               |
|                       |                      | 60         |                                |                               |
|                       |                      | 70         |                                |                               |
|                       |                      | 80         |                                |                               |
|                       |                      | 82         |                                |                               |
|                       | 8.67                 | 40         | Tandem+Lane Loading            | Truck+Lane Loading            |
|                       |                      | 50         | Truck+Lane Loading             |                               |
|                       |                      | 60         |                                |                               |
|                       |                      | 70         |                                |                               |
|                       |                      | 79         |                                |                               |



It was observed that for Standard Specifications based designs, HS-20 Truck loading always governs the moments at mid-span and shears at critical sections. For designs based on LRFD Specifications, combination of Truck and Lane loading governs for all the cases, except for 40 ft. span, where the combination of Tandem and Lane loading governs the live moments.

### **6.3.3 Undistributed Live Load Moments and Shears**

The difference in the live loads specified by the Standard and the LRFD Specifications effects the undistributed live load moments and shears. Skew and strand diameter has no effect on the undistributed live load moments or shears therefore results for cases with skew angle  $0^\circ$  and strand diameter 0.5 in. are compared in Table 6.4. The undistributed live load moments are observed to be increasing in the range of 30 percent to 48 percent for 6 ft. girder spacing when live loads based on LRFD Specifications are used as compared to the Standard Specifications. This increase was in the range of 30 percent to 45 percent for 8 ft. girder spacing and 30 percent to 44 percent for a 8.67 ft. girder spacing.

An increase was observed in the undistributed shears at critical section. The increase was found to be in the range of 9 percent to 38 percent for 6 ft. girder spacing when LRFD Specifications are used as compared to Standard Specifications. This increase was found to be in the range of 9 percent to 35 percent for 8 ft. girder spacing and 9 percent to 33 percent for 8.67 ft. girder spacing. This increase can be attributed the change in live load and also the shifting of critical section. The critical section for shear is specified by Standard specifications as  $h/2$ , where  $h$  is the depth of composite section. The LRFD Specifications requires the critical section to be calculated using an iterative process as discussed in Section 4. The difference between the undistributed moments and shears based on Standard and LRFD Specifications is found to be increasing with the increase in span length.

**Table 6.4. Comparison of Undistributed Midspan Live Load Moments and Shears at Critical Section for Type C Girder (Skew = 0°, Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Undistributed Moment (k-ft.) |        |                      | Undistributed Shear (kips) |      |                     |
|----------------------|------------|------------------------------|--------|----------------------|----------------------------|------|---------------------|
|                      |            | Standard                     | LRFD   | Difference k-ft. (%) | Standard                   | LRFD | Difference kips (%) |
| 6                    | 40         | 424.2                        | 551.6  | 127.3 (30.0)         | 50.9                       | 55.2 | 4.4 (8.6)           |
|                      | 50         | 602.4                        | 791.3  | 188.8 (31.3)         | 55.2                       | 63.8 | 8.6 (15.6)          |
|                      | 60         | 780.6                        | 1055.2 | 274.6 (35.2)         | 58.1                       | 70.7 | 12.7 (21.8)         |
|                      | 70         | 958.8                        | 1335.1 | 376.3 (39.2)         | 60.1                       | 76.6 | 16.5 (27.4)         |
|                      | 80         | 1137.0                       | 1631.1 | 494.0 (43.4)         | 61.6                       | 81.8 | 20.2 (32.8)         |
|                      | 90         | 1315.2                       | 1943.0 | 627.8 (47.7)         | 62.8                       | 86.7 | 23.9 (38.1)         |
|                      | 95         | -                            | 2105.0 | -                    | -                          | 89.2 | -                   |
|                      | 96         | 1422.4                       | -      | -                    | 63.4                       | -    | -                   |
| 8                    | 40         | 424.2                        | 551.6  | 127.3 (30.0)         | 50.9                       | 55.2 | 4.4 (8.6)           |
|                      | 50         | 602.4                        | 791.3  | 188.8 (31.3)         | 55.2                       | 64.0 | 8.8 (15.9)          |
|                      | 60         | 780.6                        | 1055.2 | 274.6 (35.2)         | 58.1                       | 70.8 | 12.7 (21.9)         |
|                      | 70         | 958.8                        | 1335.1 | 376.3 (39.2)         | 60.1                       | 76.7 | 16.6 (27.6)         |
|                      | 80         | 1137.0                       | 1631.1 | 494.0 (43.4)         | 61.6                       | 81.9 | 20.3 (33.0)         |
|                      | 83         | 1190.5                       | 1723.0 | 532.5 (44.7)         | 62.0                       | 83.5 | 21.5 (34.7)         |
| 8.67                 | 40         | 424.2                        | 551.6  | 127.3 (30.0)         | 50.9                       | 55.2 | 4.4 (8.6)           |
|                      | 50         | 602.4                        | 791.3  | 188.8 (31.3)         | 55.2                       | 64.0 | 8.7 (15.8)          |
|                      | 60         | 780.6                        | 1055.2 | 274.6 (35.2)         | 58.1                       | 70.8 | 12.7 (21.9)         |
|                      | 70         | 958.8                        | 1335.1 | 376.3 (39.2)         | 60.1                       | 76.7 | 16.6 (27.6)         |
|                      | 80         | 1137.0                       | 1631.1 | 494.0 (43.4)         | 61.6                       | 82.0 | 20.4 (33.1)         |

#### 6.3.4 Impact Factors

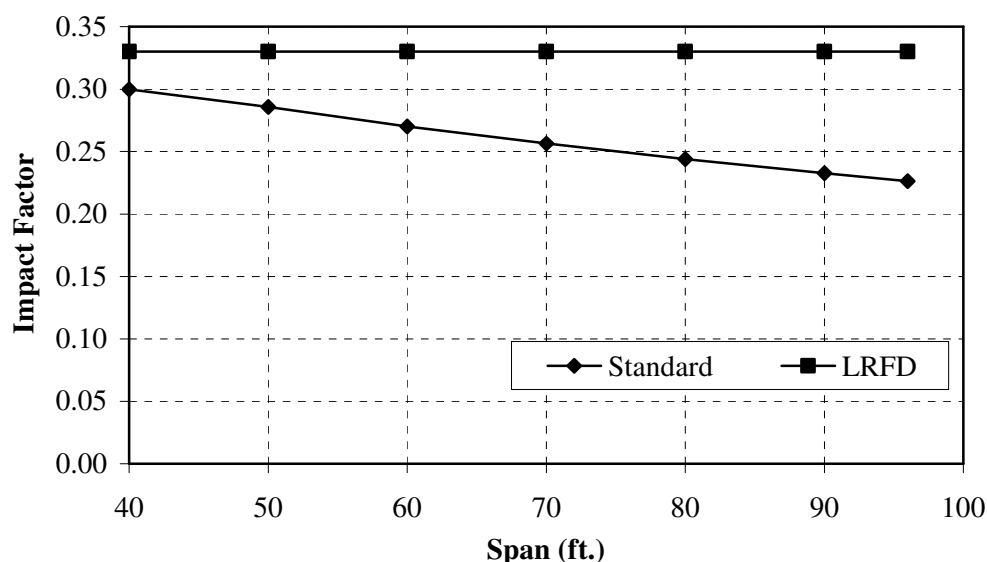
The AASHTO Standard and LRFD Specifications require that live load moments and shears be increased for impact or dynamic loading. The Standard Specifications specifies impact factors that decrease with an increase in span length, whereas the LRFD Specifications specify a constant value of dynamic loading as 33 percent of the undistributed live load moment or shear. For fatigue load moment, the LRFD Specifications specify the impact loading to be 15 percent of the undistributed live load fatigue moment. The fatigue moments are used to check the fatigue limit state required by the LRFD Specifications. The LRFD Specifications do not require the lane load moments and shears to be increased for impact loading.

A summary of impact factors and the percent difference relative to Standard value is provided in Table 6.5. The skew angle and strand diameter do not affect the impact factor, hence only the cases with skew angle  $0^\circ$  and strand diameter 0.5 in. are presented. It was observed that the LRFD Specifications provides a larger estimate of dynamic loading as compared to the Standard Specifications. This difference increases with increasing span length. The increase in the impact factor is in the range of 10 percent to 42 percent of the impact factors specified by Standard Specifications. This essentially increases the distributed live load moments for the designs based on LRFD Specifications as compared to the Standard Specifications.

**Table 6.5. Comparison of Live Load Impact Factors for Type C Girder  
(Skew =  $0^\circ$ , Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Impact Factor |      | Difference (%) |
|----------------------|------------|---------------|------|----------------|
|                      |            | Standard      | LRFD |                |
| 6                    | 40         | 0.30          | 0.33 | 10.0           |
|                      | 50         | 0.29          | 0.33 | 15.5           |
|                      | 60         | 0.27          | 0.33 | 22.1           |
|                      | 70         | 0.26          | 0.33 | 28.7           |
|                      | 80         | 0.24          | 0.33 | 35.3           |
|                      | 90         | 0.23          | 0.33 | 41.9           |
|                      | 95         | -             | 0.33 | -              |
|                      | 96         | 0.23          | -    | -              |
| 8                    | 40         | 0.30          | 0.33 | 10.0           |
|                      | 50         | 0.29          | 0.33 | 15.5           |
|                      | 60         | 0.27          | 0.33 | 22.1           |
|                      | 70         | 0.26          | 0.33 | 28.7           |
|                      | 80         | 0.24          | 0.33 | 35.3           |
|                      | 83         | 0.24          | 0.33 | 37.3           |
|                      |            |               |      |                |
| 8.67                 | 40         | 0.30          | 0.33 | 10.0           |
|                      | 50         | 0.29          | 0.33 | 15.5           |
|                      | 60         | 0.27          | 0.33 | 22.1           |
|                      | 70         | 0.26          | 0.33 | 28.7           |
|                      | 80         | 0.24          | 0.33 | 35.3           |

Figure 6.1. illustrates the impact of the LRFD Specifications on the dynamic load (impact) factors for a 6 ft. girder spacing. The same trend was observed for girder spacings of 8 ft. and 8.67 ft.



**Figure 6.1. Impact Factors for AASHTO Standard vs. AASHTO LRFD Specifications for Type C Girder**  
(Girder Spacing = 6 ft., Skew = 0°, Strand Diameter = 0.5 in)

### 6.3.5 Live Load Distribution Factors

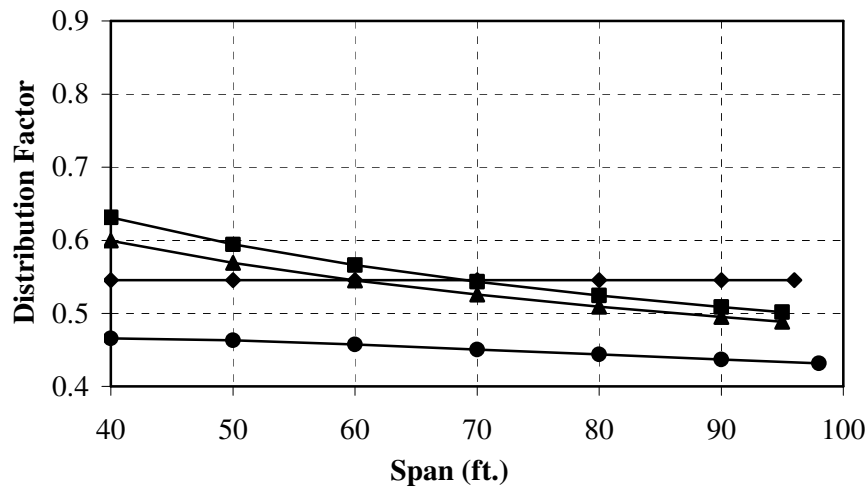
The live load moments and shears, including the dynamic (impact) load effect are distributed to the individual girders. The Standard Specifications provide a simple formula for moment distribution factor (DF) as  $S/11$  for prestressed concrete girder bridges, where  $S$  is the girder spacing in ft. The same DF is used for the distribution of live load shear to the girders. The LRFD Specifications provides more complex formulas for the distribution of live load moments and shears to individual girders. The effects of beam and slab stiffness are incorporated into these formulas. The LRFD Specifications requires the DFs for moment to be reduced and DFs for shear to be corrected for skewed bridges. Table 6.6 compares the live load moment DFs for the Standard and LRFD Specifications.

**Table 6.6. Comparison of Live Load Moment DFs (DFM) for Type C Girder  
(Strand Diameter = 0.5 in.)**

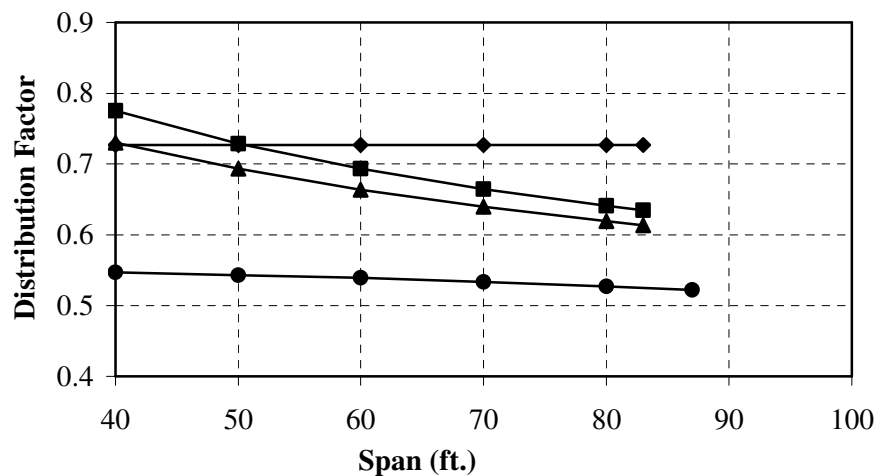
| Girder Spacing (ft.) | Span (ft.) | Std. DFM | LRFD      |         |            |         |            |         |            |         |
|----------------------|------------|----------|-----------|---------|------------|---------|------------|---------|------------|---------|
|                      |            |          | skew = 0° |         | skew = 15° |         | skew = 30° |         | skew = 60° |         |
|                      |            |          | DFM       | Diff. % | DFM        | Diff. % | DFM        | Diff. % | DFM        | Diff. % |
| 6                    | 40         | 0.545    | 0.632     | 15.8    | 0.632      | 15.8    | 0.600      | 9.9     | 0.466      | -14.6   |
|                      | 50         | 0.545    | 0.594     | 9.0     | 0.594      | 9.0     | 0.569      | 4.4     | 0.463      | -15.1   |
|                      | 60         | 0.545    | 0.566     | 3.8     | 0.566      | 3.8     | 0.545      | 0.0     | 0.457      | -16.1   |
|                      | 70         | 0.545    | 0.543     | -0.4    | 0.543      | -0.4    | 0.526      | -3.6    | 0.451      | -17.4   |
|                      | 80         | 0.545    | 0.525     | -3.8    | 0.525      | -3.8    | 0.509      | -6.7    | 0.444      | -18.6   |
|                      | 90         | 0.545    | 0.509     | -6.7    | 0.509      | -6.7    | 0.495      | -9.2    | 0.437      | -19.8   |
|                      | 95         | -        | 0.502     | -       | 0.502      | -       | 0.489      | -       | -          | -       |
|                      | 96         | 0.545    | -         | -       | -          | -       | -          | -       | -          | -       |
|                      | 98         | -        | -         | -       | -          | -       | -          | -       | 0.432      | -       |
| 8                    | 40         | 0.727    | 0.776     | 6.7     | 0.776      | 6.7     | 0.730      | 0.4     | 0.547      | -24.8   |
|                      | 50         | 0.727    | 0.729     | 0.2     | 0.729      | 0.2     | 0.693      | -4.7    | 0.543      | -25.3   |
|                      | 60         | 0.727    | 0.693     | -4.7    | 0.693      | -4.7    | 0.664      | -8.8    | 0.540      | -25.8   |
|                      | 70         | 0.727    | 0.665     | -8.6    | 0.665      | -8.6    | 0.639      | -12.1   | 0.534      | -26.6   |
|                      | 80         | 0.727    | 0.641     | -11.9   | 0.641      | -11.9   | 0.619      | -14.9   | 0.527      | -27.5   |
|                      | 83         | 0.727    | 0.635     | -12.7   | 0.635      | -12.7   | 0.614      | -15.6   | -          | -       |
|                      | 87         | -        | -         | -       | -          | -       | -          | -       | 0.522      | -       |
| 8.67                 | 40         | 0.788    | 0.822     | 4.3     | 0.822      | 4.3     | 0.772      | -2.0    | 0.573      | -27.3   |
|                      | 50         | 0.788    | 0.772     | -2.0    | 0.772      | -2.0    | 0.733      | -7.0    | 0.567      | -28.0   |
|                      | 60         | 0.788    | 0.734     | -6.8    | 0.734      | -6.8    | 0.702      | -11.0   | 0.565      | -28.3   |
|                      | 70         | 0.788    | 0.704     | -10.7   | 0.704      | -10.7   | 0.676      | -14.2   | 0.560      | -29.0   |
|                      | 80         | 0.788    | 0.679     | -13.9   | 0.679      | -13.9   | 0.654      | -17.0   | 0.553      | -       |
|                      | 81         | -        | -         | -       | -          | -       | 0.653      | -       | -          | -       |
|                      | 85         | -        | -         | -       | -          | -       | -          | -       | 0.550      | -       |

It was observed that the live load moment DFs given by the LRFD Specifications are smaller as compared to those for the Standard Specifications for most of the cases. The difference increases with an increase in span length because the LRFD DFs decrease with an increase in the span while span length has no effect on the Standard DFs. The moment DFs increase with increase in girder spacing for both the AASHTO Standard and LRFD Specifications. In addition, the difference between the DFs increased for

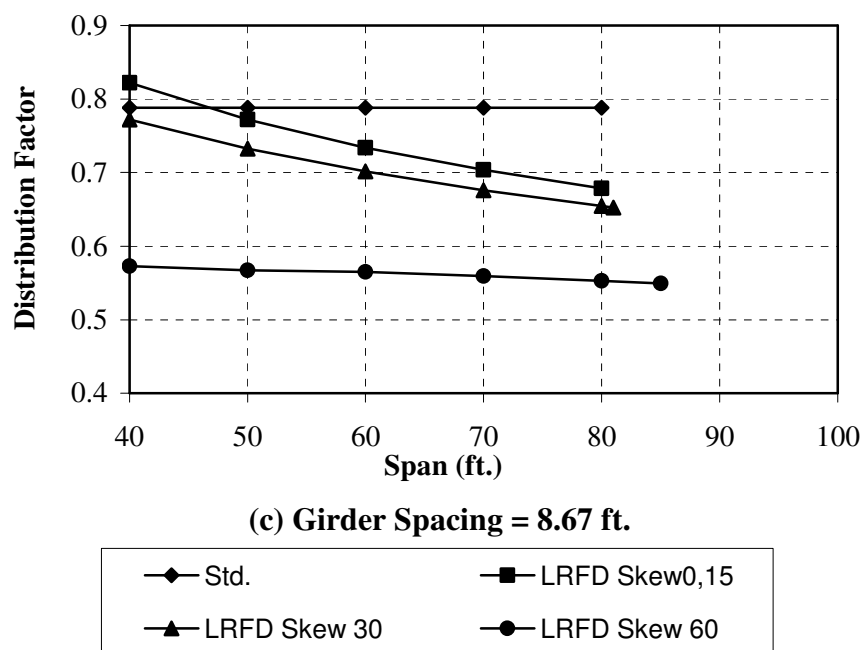
larger girder spacings. The LRFD live load moment DFs are the same for  $0^\circ$  and  $15^\circ$  skews, but there is a significant change when the skew angles are  $30^\circ$  and  $60^\circ$ . It was observed that increase in skew angles beyond  $30^\circ$  decreases the moment DFs significantly for Type C girder bridges. The maximum difference between the Standard and LRFD DFs was found to be 16 percent for 6ft. girder spacing, 14 percent for 8 ft and 8.67 ft. girder spacing for the skew angle of  $0^\circ$ . This difference increased to 20 percent for 6 ft., 28 percent for 8 ft. and 30 percent for 8.67 ft. girder spacing for a skew angle of  $60^\circ$ . Figure 6.3 shows the effect of skew on the moment DFs for 6 ft., 8 ft. and 8.67 ft. girder spacing.



(a) Girder Spacing = 6 ft.



(b) Girder Spacing = 8 ft.



**Figure 6.2. Comparison of Live Load Moment DFs by Girder Spacing for Type C Girder (Strand Dia. = 0.5 in.)**

Table 6.7 and Figure 6.4 provide a summary of shear DFs for the parametric study with Type C girders. The strand diameter does not affect the DFs for shear.

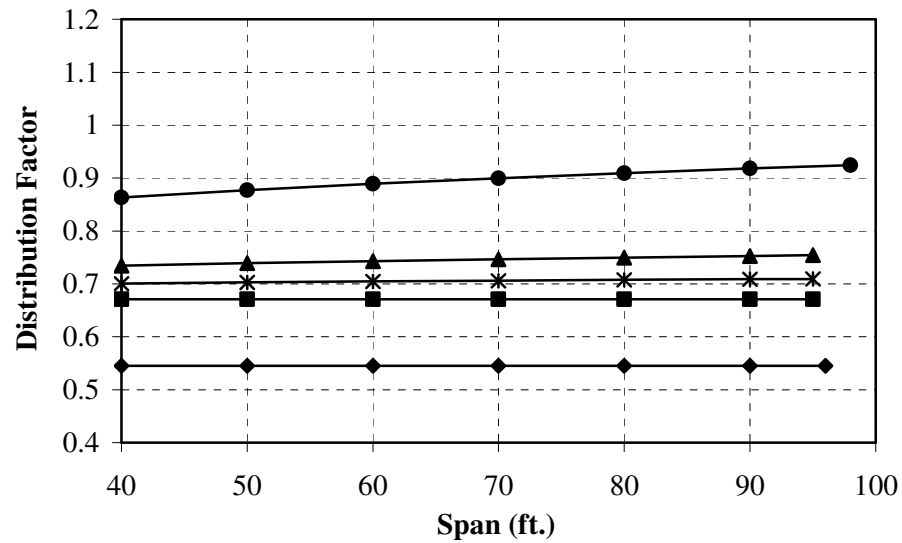
**Table 6.7. Comparison of Live Load Shear DFs (DFV) for Type C Girder  
(Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. DFV | LRFD      |         |            |         |            |         |            |         |
|----------------------|------------|----------|-----------|---------|------------|---------|------------|---------|------------|---------|
|                      |            |          | skew = 0° |         | skew = 15° |         | skew = 30° |         | skew = 60° |         |
|                      |            |          | DFV       | Diff. % | DFV        | Diff. % | DFV        | Diff. % | DFV        | Diff. % |
| 6                    | 40         | 0.545    | 0.671     | 22.9    | 0.700      | 28.4    | 0.735      | 34.7    | 0.863      | 58.3    |
|                      | 50         | 0.545    | 0.671     | 22.9    | 0.703      | 28.8    | 0.739      | 35.6    | 0.877      | 60.8    |
|                      | 60         | 0.545    | 0.671     | 22.9    | 0.704      | 29.1    | 0.743      | 36.3    | 0.889      | 63.0    |
|                      | 70         | 0.545    | 0.671     | 22.9    | 0.706      | 29.4    | 0.747      | 37.0    | 0.900      | 65.0    |
|                      | 80         | 0.545    | 0.671     | 22.9    | 0.708      | 29.7    | 0.750      | 37.5    | 0.909      | 66.7    |
|                      | 90         | 0.545    | 0.671     | 22.9    | 0.709      | 30.0    | 0.753      | 38.1    | 0.918      | 68.3    |
|                      | 95         | -        | 0.671     | -       | 0.710      | -       | 0.754      | -       | -          | -       |
|                      | 96         | 0.545    | -         | -       | -          | -       | -          | -       | -          | -       |
|                      | 98         | -        | -         | -       | -          | -       | -          | -       | 0.925      | -       |
| 8                    | 40         | 0.727    | 0.814     | 12.0    | 0.851      | 17.0    | 0.892      | 22.7    | 1.049      | 44.2    |
|                      | 50         | 0.727    | 0.814     | 12.0    | 0.853      | 17.3    | 0.898      | 23.5    | 1.065      | 46.5    |
|                      | 60         | 0.727    | 0.814     | 12.0    | 0.855      | 17.6    | 0.903      | 24.1    | 1.080      | 48.5    |
|                      | 70         | 0.727    | 0.814     | 12.0    | 0.857      | 17.9    | 0.907      | 24.7    | 1.093      | 50.2    |
|                      | 80         | 0.727    | 0.814     | 12.0    | 0.859      | 18.2    | 0.911      | 25.3    | 1.104      | 51.8    |
|                      | 83         | 0.727    | 0.814     | 12.0    | 0.860      | 18.2    | 0.912      | 25.4    | -          | -       |
|                      | 87         | -        | -         | -       | -          | -       | -          | -       | 1.112      | -       |
| 8.67                 | 40         | 0.788    | 0.861     | 9.3     | 0.899      | 14.1    | 0.944      | 19.7    | 1.109      | 40.7    |
|                      | 50         | 0.788    | 0.861     | 9.3     | 0.902      | 14.5    | 0.950      | 20.5    | 1.127      | 42.9    |
|                      | 60         | 0.788    | 0.861     | 9.3     | 0.905      | 14.8    | 0.955      | 21.1    | 1.142      | 44.9    |
|                      | 70         | 0.788    | 0.861     | 9.3     | 0.907      | 15.0    | 0.959      | 21.7    | 1.155      | 46.6    |
|                      | 80         | 0.788    | 0.861     | 9.3     | 0.909      | 15.3    | 0.963      | 22.2    | 1.168      | -       |
|                      | 81         | -        | -         | -       | -          | -       | 0.964      | -       | -          | -       |
|                      | 85         | -        | -         | -       | -          | -       | -          | -       | 1.173      | -       |

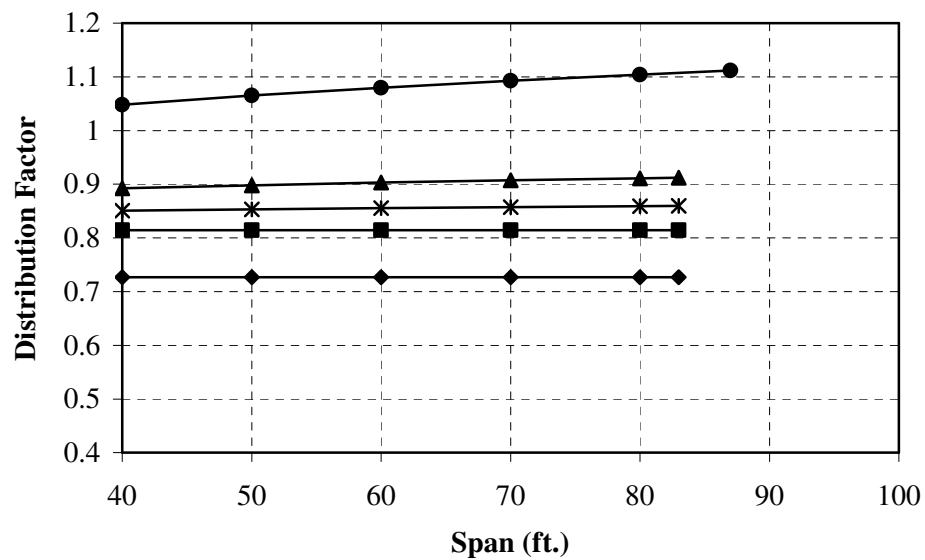
The LRFD live load shear DFs specified by LRFD Specifications are larger as compared to the Standard Specifications. The DFs increases with an increase in girder spacing for both specifications and the LRFD DFs approach Standard DFs as the girder spacing is increased. The span length and skew angle has no impact on the shear distribution factors specified by Standard Specifications. The maximum difference in the



shear distribution factors is found to be 68 percent for 6 ft. spacing, 52 percent for 8 ft. spacing and 47 percent for the 8.67 ft. spacing.

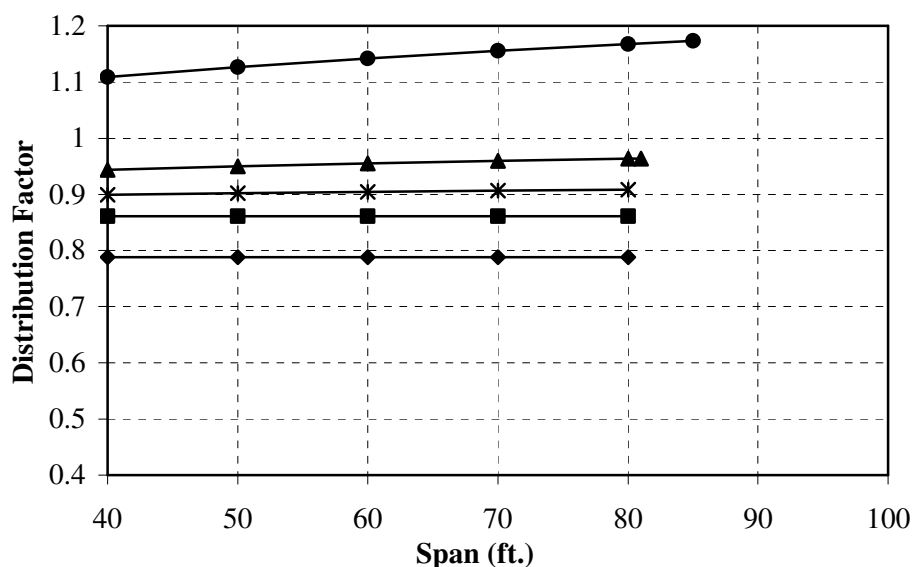


(a) Girder Spacing = 6 ft.

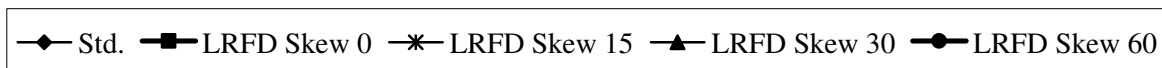


(b) Girder Spacing = 8 ft.

**Figure 6.3. Comparison of Live Load Shear DFs for Type C Girder (Strand Dia. = 0.5 in.)**



(c) Girder Spacing = 8.67 ft.



**Figure 6.3. (Cont.) Comparison of Live Load Shear DFs for Type C Girder  
(Strand Dia. = 0.5 in.)**

### 6.3.6 Distributed Live Load Moments and Shears

The combined effect of the impact and distribution factors on the live load moment and shears is presented in this section. The distributed live load moments are compared in Table 6.8. The distributed live load moments are the same for 0° and 15° skew angles for LRFD Specifications because the distribution factors for these two skews are identical. The distributed live load moments were found to be significantly larger than those for the Standard Specifications. The distributed live load moments increase in the range of 37 percent to 45 percent for 6 ft. girder spacing when the skew angle is 0°. As the girder spacing increases the difference between the distributed live load moments decreases. The LRFD moments were found to be in the range of 25 percent to 34 percent larger for 8 ft. and 22 percent to 31 percent larger for 8.67 ft. girder spacing when the skew angle is 0°.

The increase in skew angle for and beyond 30° results in decrease in distribution factors. This causes the live load moments to decrease. The difference between the live load moments for Standard and LRFD Specifications was found to be in the range of 7 percent to 17 percent for 6 ft. girder spacing when skew angle is 60°. This difference reduces to the range of -7 percent to 2.5 percent and -11 percent to -3 percent for 8 ft. and 8.67 ft. girder spacing respectively. The increase in span length increases the difference between the live load moments specified by the two codes.

**Table 6.8. Comparison of Distributed Midspan Live Load Moments (LL Mom.) for Standard and LRFD Specifications for Type C Girder (Strand Dia. = 0.5 in.)**

[illegible]

The distributed shear force at the critical section due to live load is found to be increasing significantly when LRFD Specifications are used. The increase in the shear force can be attributed to the increase in the undistributed shear force due to HL93 loading and the increase in distribution factors. The shear force at the critical section for LRFD Specifications is found to be increasing in the range of 53 percent to 140 percent for 6 ft. girder spacing as compared to the Standard specifications. The increase was found to be in the range of 33 percent to 110 percent for 8 ft. and 30 percent to 95 percent for 8.67 ft. girder spacing. The results are presented in Table 6.9.

**Table 6.9. Comparison of Distributed Live Load Shear at Critical Section for Standard and LRFD Specifications for Type C Girder (Strand Dia. = 0.5 in.)**

[illegible]

## **6.4 IMPACT OF AASHTO LRFD SPECIFICATIONS ON SERVICE LOAD DESIGN**

### **6.4.1 General**

The impact of LRFD Specifications on the service load design results is discussed in this section. The effect on prestress losses, required number of strands and the required concrete strengths at service and at release is discussed. The increase in the live load moment and the change in equations for prestress loss calculations specified by AASHTO LRFD Specifications results in different service load design requirements. The change in the service load combination and allowable stress limits also affects the design. Generally the design requirements for LRFD Specifications were found to be conservative as compared to Standard Specifications.

### **6.4.2 Impact on Prestress Losses**

The loss in prestress occurs mainly from four sources viz. shrinkage of concrete, relaxation of steel, elastic shortening of steel and creep of concrete. These losses are categorized into initial prestress loss and final prestress loss. The initial prestress loss occurs due to initial relaxation of steel and elastic shortening of prestressing strands. The final loss occurs due to final relaxation, creep and shrinkage of concrete. Total prestress loss is the combination of initial and final loss.

#### *6.4.2.1 Initial Prestress Loss*

The initial prestress loss is the combination of the losses due to elastic shortening and initial steel relaxation. The combined effect of the changes in these two losses was observed in the initial loss percent calculations between Standard and LRFD Specifications. The initial loss estimates provided by LRFD Specifications are found to be conservative as compared to the Standard Specifications.

**Table 6.10. Comparison of Initial Prestress Loss (%) for AASHTO Standard and LRFD Specifications for Type C Girder (Strand Dia. = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. Initial Loss (%) | LRFD           |         |                |         |                |         |                |         |
|----------------------|------------|-----------------------|----------------|---------|----------------|---------|----------------|---------|----------------|---------|
|                      |            |                       | skew = 0°      |         | skew = 15°     |         | skew = 30°     |         | skew = 60°     |         |
|                      |            |                       | Init. Loss (%) | Diff. % | Init. Loss (%) | Diff. % | Init. Loss (%) | Diff. % | Init. Loss (%) | Diff. % |
| 6                    | 40         | 6.5                   | 6.2            | -5.3    | 6.2            | -5.3    | 6.2            | -5.3    | 6.2            | -5.3    |
|                      | 50         | 6.3                   | 6.6            | 5.0     | 6.6            | 5.0     | 6.6            | 5.0     | 6.6            | 5.0     |
|                      | 60         | 7.0                   | 7.4            | 6.0     | 7.4            | 6.0     | 7.4            | 5.7     | 7.4            | 5.7     |
|                      | 70         | 7.9                   | 8.5            | 6.6     | 8.5            | 6.6     | 8.5            | 6.6     | 8.0            | 1.0     |
|                      | 80         | 8.6                   | 9.5            | 10.7    | 9.5            | 10.7    | 9.5            | 10.7    | 9.2            | 7.1     |
|                      | 90         | 9.5                   | 10.5           | 10.3    | 10.5           | 10.3    | 10.4           | 10.3    | 10.2           | 7.9     |
|                      | 95         | -                     | 11.0           | -       | 11.0           | -       | 10.9           | -       | -              | -       |
|                      | 96         | 10.0                  | -              | -       | -              | -       | -              | -       | -              | -       |
|                      | 98         | -                     | -              | -       | -              | -       | -              | -       | 11.0           | -       |
| 8                    | 40         | 7.0                   | 6.9            | -1.3    | 6.9            | -1.3    | 6.9            | -1.3    | 6.2            | -11.6   |
|                      | 50         | 6.8                   | 7.7            | 13.0    | 7.7            | 13.0    | 7.2            | 5.6     | 6.6            | -2.8    |
|                      | 60         | 7.8                   | 8.3            | 6.5     | 8.3            | 6.5     | 8.3            | 6.5     | 7.8            | 0.4     |
|                      | 70         | 8.8                   | 9.4            | 7.4     | 9.4            | 7.4     | 9.4            | 7.4     | 9.1            | 3.9     |
|                      | 80         | 9.7                   | 10.4           | 8.2     | 10.4           | 8.2     | 10.4           | 8.2     | 10.2           | 5.9     |
|                      | 83         | 9.9                   | 10.9           | 10.0    | 10.9           | 10.0    | 10.8           | 8.4     | -              | -       |
|                      | 87         | -                     | -              | -       | -              | -       | -              | -       | 10.9           | -       |
| 8.67                 | 40         | 7.0                   | 6.9            | -1.3    | 6.9            | -1.3    | 6.9            | -1.3    | 6.2            | -11.7   |
|                      | 50         | 7.2                   | 7.7            | 6.1     | 7.7            | 6.1     | 7.7            | 6.1     | 7.2            | -0.9    |
|                      | 60         | 7.8                   | 8.3            | 6.4     | 8.3            | 6.4     | 8.3            | 6.4     | 7.9            | 0.7     |
|                      | 70         | 9.0                   | 9.7            | 7.6     | 9.7            | 7.6     | 9.7            | 7.6     | 9.4            | 4.4     |
|                      | 80         | 10.0                  | 10.8           | 8.6     | 10.8           | 8.6     | 10.8           | 8.5     | 10.4           | -       |
|                      | 81         | -                     | -              | -       | -              | -       | 11.0           | -       | -              | -       |
|                      | 85         | -                     | -              | -       | -              | -       | -              | -       | 11.0           | -       |

Except for few cases when skew angle was 60°, the increase in the initial loss percent was found to be in the range of 5 percent to 11 percent. The skew angle of 30° does not have a significant effect on initial loss percent however, the skew angle of 60° was found to decrease the initial loss percent significantly. This trend follows the trend of the loss due to elastic shortening as it is the major contributor to the initial losses. Table 6.10 presents the results for strand diameter of 0.5 in. Similar trends were observed for 0.6 in. diameter strands.



The conservatism was found to be decreasing with the increase in span length, skew angle and girder spacing. The results for 0.5 in. diameter strands are presented in Table 6.11.

#### **6.4.3. Impact on the Required Number of Prestressing Strands**

The number of strands required depends on the allowable stress limits and the stresses caused due to dead load and live load. There is a change in the allowable stress limits in LRFD Specifications and the live load stresses are also different. The Service III limit state that checks the bottom tensile stresses also impacts the prestressing strand requirements. The difference in the prestress losses is another factor that effects the final strand requirements. Strength limit state controls the number of strands for case when span length is lesser than 60 ft.

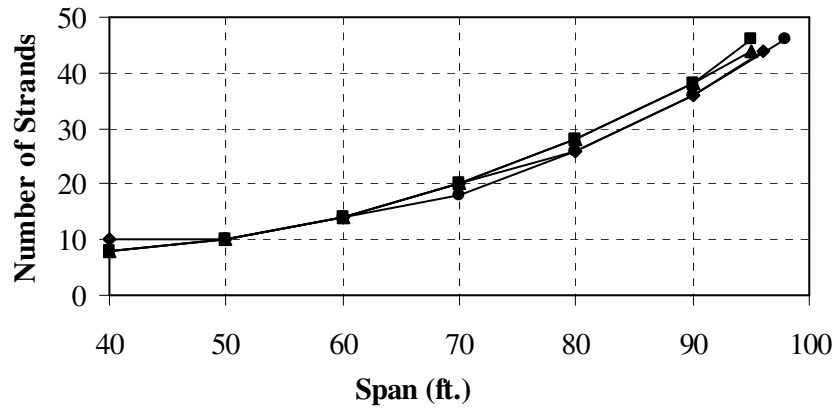
The LRFD Specifications require the same number of strands as Standard Specifications for most of the cases for 0.5 in diameter strands. For the cases with skew angle of  $60^\circ$  the number of strands required by LRFD Specifications was found to be lesser than that of Standard Specifications. The difference was found to be in the range of 2 to 4 strands. The results for 0.5 in diameter strands are presented in Table 6.16. Similar trends were found for 0.6 in. diameter strands. The results for 0.6 in. diameter strands are presented in Table 6.17. Figures 6.4 illustrates the comparison of strand requirement for Standard and LRFD Specifications.



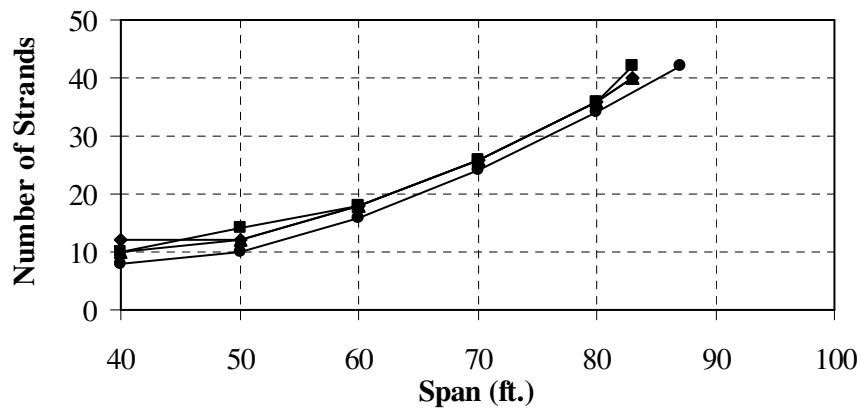
**Table 6.12. Comparison of Required Number of Strands for AASHTO Standard and LRFD Specifications for Type C Girder (Strand Dia. = 0.5 in.)**

[illegible]

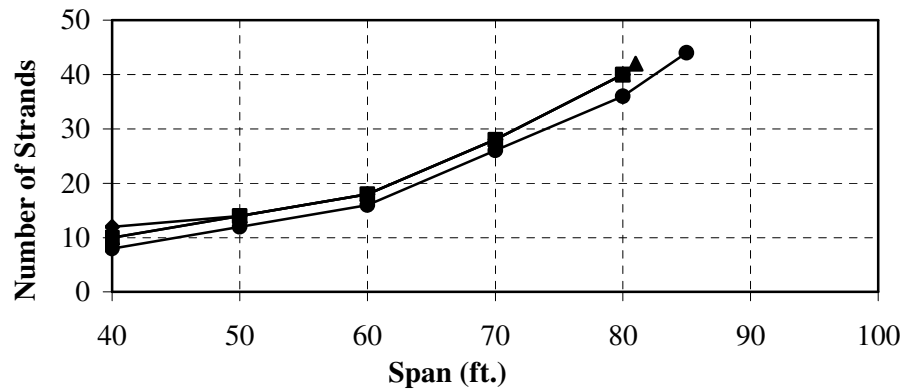




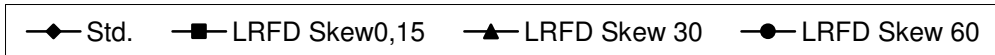
(a) Girder Spacing = 6 ft.



(b) Girder Spacing = 8 ft.



(c) Girder Spacing = 8.67 ft.



**Figure 6.4. Comparison of Required Number of Strands for AASHTO Standard and LRFD Specifications for Type C Girder (Strand Dia. = 0.5 in.)**



The minimum strength at release was considered to be 4000 psi. For span lengths less than 70 ft., it was observed that minimum concrete strength governs. The LRFD Specifications yields a slightly conservative estimate of the concrete strength at release. The maximum difference was found to be 8 percent. The concrete strength at release is limited to 6750 psi. and in most of the cases it governs the maximum span length. The results for 0.5 in. diameter strands are presented in Table 6.14.

#### *6.4.4.2 Concrete Strength at Service*

The concrete strength at service is affected by the stresses at the midspan due to prestressing force, dead loads, superimposed loads and live loads. The concrete strength at service is limited to 8750 psi. However, this limitation does not affect the maximum span length as the initial concrete strength approaches its limits earlier than the final concrete strength. The minimum strength was considered as 5000 psi and for span lengths less than 80 ft. it was observed that this limit controls. Also the concrete strength at service cannot be smaller than the concrete strength at release. This limitation governs for a few cases for 0.5 in diameter strands and several cases for 0.6 in. diameter strands.

The LRFD Specifications do not have a significant effect on the concrete strength at service. A small reduction in the required concrete strength was observed for few cases, maximum difference being 10 percent. The results for 0.5 in. diameter strands are presented in Table 6.15.

**Table 6.15. Comparison of Concrete Strength at Service ( $f'_c$ ) for AASHTO Standard and LRFD Specifications for Type C Girder (Strand Dia. = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. $f'_c$ (psi) | LRFD         |         |              |         |              |         |              |         |
|----------------------|------------|-------------------|--------------|---------|--------------|---------|--------------|---------|--------------|---------|
|                      |            |                   | skew = 0°    |         | skew = 15°   |         | skew = 30°   |         | skew = 60°   |         |
|                      |            |                   | $f'_c$ (psi) | Diff. % | $f'_c$ (psi) | Diff. % | $f'_c$ (psi) | Diff. % | $f'_c$ (psi) | Diff. % |
| 6                    | 40         | 5000.0            | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
|                      | 50         | 5000.0            | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
|                      | 60         | 5000.0            | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
|                      | 70         | 5000.0            | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
|                      | 80         | 5000.0            | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
|                      | 90         | 6375.5            | 6526.0       | 2.4     | 6526.0       | 2.4     | 5935.7       | -6.9    | 5702.7       | -10.6   |
|                      | 95         | -                 | 6886.5       | -       | 6886.5       | -       | 8012.2       | -       | -            | -       |
|                      | 96         | 7754.7            | -            | -       | -            | -       | -            | -       | -            | -       |
|                      | 98         | -                 | -            | -       | -            | -       | -            | -       | 7877.3       | -       |
| 8                    | 40         | 5000.0            | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
|                      | 50         | 5000.0            | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
|                      | 60         | 5000.0            | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
|                      | 70         | 5000.0            | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
|                      | 80         | 6088.5            | 6804.5       | 11.8    | 6804.5       | 11.8    | 6021.1       | -1.1    | 5678.2       | -6.7    |
|                      | 83         | 6624.5            | 6852.9       | 3.4     | 6852.9       | 3.4     | 6546.8       | -1.2    | -            | -       |
|                      | 87         | -                 | -            | -       | -            | -       | -            | -       | 7344.9       | -       |
| 8.67                 | 40         | 5000.0            | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
|                      | 50         | 5000.0            | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     | 5000.0       | 0.0     |
|                      | 60         | 5000.0            | 5177.9       | 3.6     | 5177.9       | 3.6     | 5000.0       | 0.0     | 5000.0       | 0.0     |
|                      | 70         | 5029.6            | 5000.0       | -0.6    | 5000.0       | -0.6    | 5000.0       | -0.6    | 5000.0       | -0.6    |
|                      | 80         | 6749.1            | 6669.9       | -1.2    | 6669.9       | -1.2    | 6670.2       | -1.2    | 6021.4       | -       |
|                      | 81         | -                 | -            | -       | -            | -       | 6935.6       | -       | -            | -       |
|                      | 85         | -                 | -            | -       | -            | -       | -            | -       | 7073.0       | -       |

#### 6.4.5 Impact of AASHTO LRFD on Maximum Span Length

The maximum span lengths are limited by the concrete strength at release of 6750 psi and concrete strength at service of 8750 psi. The maximum span is not governed by maximum number of strands for any of the cases considered for the parametric study. The maximum allowable concrete strengths are reached when the strand number was in the range of 42 to 44, whereas Type C girder can hold up to 74 strands. Thus by relaxing the limit on concrete strengths longer spans can be achieved.

The LRFD Specifications have a reducing effect on the maximum span length in few cases and no effect in others when the skew angle is less than 60 degrees. This is due to a slightly conservative estimate of the concrete strengths which reaches the limits earlier than the Standard Specifications. The maximum span length for skew angle of 60 degrees is larger as compared to those possible by Standard Specifications. However, the difference between the maximum span lengths was found to be negligible. The difference was in the range of -3 percent to 6 percent for all the cases with strand diameter of 0.5 in. and 0.6 in. The results for maximum span length are presented in Table 6.16.

**Table 6.16. Comparison of Maximum Span Lengths for AASHTO Standard and LRFD Specifications for Type C Girder**

| Strand Dia. (in.) | Girder Spacing (ft.) | Std. Max. Span (ft.) | LRFD            |           |                 |           |                 |           |                 |           |
|-------------------|----------------------|----------------------|-----------------|-----------|-----------------|-----------|-----------------|-----------|-----------------|-----------|
|                   |                      |                      | Skew 0°         |           | Skew 15°        |           | Skew 30°        |           | Skew 60°        |           |
|                   |                      |                      | Max. Span (ft.) | Diff. (%) | Max. Span (ft.) | Diff. (%) | Max. Span (ft.) | Diff. (%) | Max. Span (ft.) | Diff. (%) |
| 0.5               | 6                    | 96                   | 95              | -1.0      | 95              | -1.0      | 95              | -1.0      | 98              | 2.1       |
|                   | 8                    | 83                   | 83              | 0.0       | 83              | 0.0       | 83              | 0.0       | 87              | 4.8       |
|                   | 8.67                 | 80                   | 80              | 0.0       | 80              | 0.0       | 81              | 1.3       | 85              | 6.3       |
| 0.6               | 6                    | 95                   | 92              | -3.2      | 92              | -3.2      | 93              | -2.1      | 96              | 1.1       |
|                   | 8                    | 82                   | 82              | 0.0       | 82              | 0.0       | 83              | 1.2       | 87              | 6.1       |
|                   | 8.67                 | 79                   | 79              | 0.0       | 79              | 0.0       | 80              | 1.3       | 83              | 5.1       |

## 6.5 IMPACT OF AASHTO LRFD SPECIFICATIONS ON FLEXURAL STRENGTH LIMIT STATE

### 6.5.1 General

The impact of LRFD Specifications on the requirements of ultimate flexural strength limit state design is discussed in following section. The change in the load combination, the required concrete strengths and the number of strands from service limit state effects the ultimate flexural strength limit. The impact of change in the definition of the rectangular section behavior and flanged section behavior in LRFD

Specifications is discussed. The reinforcement limits have also been changed in the LRFD specifications, however for all the cases of Standard and LRFD Specifications the sections are found to be under reinforced.

### **6.5.2 Impact on Design Moment**

The load combinations for ultimate limit state are significantly changed from Standard to LRFD Specifications. The load factors for moments due to live load and dead loads except wearing surface load specified by LRFD Specifications are smaller than the Standard Specifications. The load factor for moment due to wearing surface load is increased in the LRFD Specifications. The live load moments specified by LRFD specifications are larger than that of Standard Specifications. The combined effect of these two changes results in the design moments that are comparable.

The LRFD Specifications yields design moments that are in general slightly smaller as compared to the Standard Specifications. The difference is found to decrease with the increase in span length, girder spacing and skew angle beyond 30°. The difference in the design moments was found to be in the range of -25 percent to 7 percent for 6 ft., -25 percent to 2 percent for 8 ft. and -25 percent to 8 percent for 8.67 ft. girder spacing. The strand diameter does not have any effect on the design moments. The comparison of the design moments specified by Standard and LRFD specifications is presented in Table 6.17.



**Table 6.17. Comparison of Factored Ultimate Moment ( $M_u$ ) for AASHTO Standard and LRFD Specifications for Type C Girder (Strand Dia. = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. $M_u$ (k-ft.) | LRFD          |         |               |         |               |         |               |         |
|----------------------|------------|--------------------|---------------|---------|---------------|---------|---------------|---------|---------------|---------|
|                      |            |                    | skew = 0°     |         | skew = 15°    |         | skew = 30°    |         | skew = 60°    |         |
|                      |            |                    | $M_u$ (k-ft.) | Diff. % | $M_u$ (k-ft.) | Diff. % | $M_u$ (k-ft.) | Diff. % | $M_u$ (k-ft.) | Diff. % |
| 6                    | 40         | 1167.1             | 1072.3        | -8.1    | 1072.3        | -8.1    | 1033.6        | -11.4   | 870.9         | -25.4   |
|                      | 50         | 1415.1             | 1513.4        | 6.9     | 1513.4        | 6.9     | 1469.6        | 3.9     | 1286.0        | -9.1    |
|                      | 60         | 1897.7             | 2003.5        | 5.6     | 2003.5        | 5.6     | 1955.4        | 3.0     | 1753.9        | -7.6    |
|                      | 70         | 2417.6             | 2533.9        | 4.8     | 2533.9        | 4.8     | 2482.4        | 2.7     | 2266.2        | -6.3    |
|                      | 80         | 2975.4             | 3106.9        | 4.4     | 3106.9        | 4.4     | 3052.3        | 2.6     | 2823.3        | -5.1    |
|                      | 90         | 3571.8             | 3723.7        | 4.3     | 3723.7        | 4.3     | 3666.3        | 2.6     | 3425.8        | -4.1    |
|                      | 95         | -                  | 4048.8        | -       | 4048.8        | -       | 3990.2        | -       | -             | -       |
|                      | 96         | 3948.6             | -             | -       | -             | -       | -             | -       | -             | -       |
|                      | 98         | -                  | -             | -       | -             | -       | -             | -       | 3941.4        | -       |
| 8                    | 40         | 1360.7             | 1309.0        | -3.8    | 1309.0        | -3.8    | 1254.0        | -7.8    | 1023.4        | -24.8   |
|                      | 50         | 1820.9             | 1844.0        | 1.3     | 1844.0        | 1.3     | 1782.0        | -2.1    | 1522.0        | -16.4   |
|                      | 60         | 2434.4             | 2437.5        | 0.1     | 2437.5        | 0.1     | 2369.6        | -2.7    | 2084.7        | -14.4   |
|                      | 70         | 3092.0             | 3079.0        | -0.4    | 3079.0        | -0.4    | 3006.2        | -2.8    | 2700.8        | -12.7   |
|                      | 80         | 3794.6             | 3771.1        | -0.6    | 3771.1        | -0.6    | 3694.1        | -2.6    | 3371.0        | -11.2   |
|                      | 83         | 4014.3             | 3988.9        | -0.6    | 3988.9        | -0.6    | 3910.7        | -2.6    | -             | -       |
|                      | 87         | -                  | -             | -       | -             | -       | -             | -       | 3872.5        | -       |
| 8.67                 | 40         | 1420.8             | 1381.1        | -2.8    | 1381.1        | -2.8    | 1320.5        | -7.1    | 1065.9        | -25.0   |
|                      | 50         | 1804.7             | 1944.0        | 7.7     | 1944.0        | 7.7     | 1875.7        | 3.9     | 1588.8        | -12.0   |
|                      | 60         | 2603.1             | 2567.8        | -1.4    | 2567.8        | -1.4    | 2492.9        | -4.2    | 2178.7        | -16.3   |
|                      | 70         | 3302.7             | 3241.5        | -1.9    | 3241.5        | -1.9    | 3161.3        | -4.3    | 2824.6        | -14.5   |
|                      | 80         | 4049.2             | 3967.8        | -2.0    | 3967.8        | -2.0    | 3883.0        | -4.1    | 3526.9        | -       |
|                      | 81         | -                  | -             | -       | -             | -       | 3958.1        | -       | -             | -       |
|                      | 85         | -                  | -             | -       | -             | -       | -             | -       | 3899.3        | -       |

### 6.5.3 Impact on Moment Resistance

The change in the concrete strength at service and the number of strands affects the moment resistance capacity of the section. The change in expression for evaluation of effective prestress in the prestressing strands also has an effect on the ultimate moment resistance of the section.

**Table 6.18. Comparison of Moment Resistance ( $M_r$ ) for AASHTO Standard and LRFD Specifications for Type C Girder (Strand Dia. = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. $M_r$ (k-ft.) | LRFD          |         |               |         |               |         |               |         |
|----------------------|------------|--------------------|---------------|---------|---------------|---------|---------------|---------|---------------|---------|
|                      |            |                    | skew = 0°     |         | skew = 15°    |         | skew = 30°    |         | skew = 60°    |         |
|                      |            |                    | $M_r$ (k-ft.) | Diff. % | $M_r$ (k-ft.) | Diff. % | $M_r$ (k-ft.) | Diff. % | $M_r$ (k-ft.) | Diff. % |
| 6                    | 40         | 1228.2             | 1236.5        | 0.7     | 1236.5        | 0.7     | 1236.5        | 0.7     | 1236.5        | 0.7     |
|                      | 50         | 1538.3             | 1536.3        | -0.1    | 1536.3        | -0.1    | 1536.3        | -0.1    | 1536.3        | -0.1    |
|                      | 60         | 2101.2             | 2097.5        | -0.2    | 2097.5        | -0.2    | 2097.5        | -0.2    | 2097.5        | -0.2    |
|                      | 70         | 2919.6             | 2912.9        | -0.2    | 2912.9        | -0.2    | 2912.9        | -0.2    | 2644.6        | -9.4    |
|                      | 80         | 3666.5             | 3897.5        | 6.3     | 3897.5        | 6.3     | 3897.5        | 6.3     | 3656.3        | -0.3    |
|                      | 90         | 4805.1             | 5001.1        | 4.1     | 5001.1        | 4.1     | 5001.1        | 4.1     | 4789.3        | -0.3    |
|                      | 95         | -                  | 5601.9        | -       | 5601.9        | -       | 5469.9        | -       | -             | -       |
|                      | 96         | 5596.9             | -             | -       | -             | -       | -             | -       | -             | -       |
|                      | 98         | -                  | -             | -       | -             | -       | -             | -       | 5603.0        | -       |
| 8                    | 40         | 1587.0             | 1547.9        | -2.5    | 1547.9        | -2.5    | 1547.9        | -2.5    | 1244.0        | -21.6   |
|                      | 50         | 1837.5             | 2120.0        | 15.4    | 2120.0        | 15.4    | 1835.4        | -0.1    | 1547.9        | -15.8   |
|                      | 60         | 2685.8             | 2681.3        | -0.2    | 2681.3        | -0.2    | 2681.3        | -0.2    | 2402.0        | -10.6   |
|                      | 70         | 3739.3             | 3730.7        | -0.2    | 3730.7        | -0.2    | 3730.7        | -0.2    | 3475.7        | -7.0    |
|                      | 80         | 4940.9             | 4926.8        | -0.3    | 4926.8        | -0.3    | 4926.8        | -0.3    | 4698.0        | -4.9    |
|                      | 83         | 5379.7             | 5571.2        | 3.6     | 5571.2        | 3.6     | 5363.3        | -0.3    | -             | -       |
|                      | 87         | -                  | -             | -       | -             | -       | -             | -       | 5571.2        | -       |
| 8.67                 | 40         | 1482.6             | 1550.6        | 4.6     | 1550.6        | 4.6     | 1550.6        | 4.6     | 1245.7        | -16.0   |
|                      | 50         | 1984.8             | 2125.3        | 7.1     | 2125.3        | 7.1     | 2125.3        | 7.1     | 1839.2        | -7.3    |
|                      | 60         | 2694.1             | 2689.9        | -0.2    | 2689.9        | -0.2    | 2689.9        | -0.2    | 2408.8        | -10.6   |
|                      | 70         | 4012.5             | 4003.4        | -0.2    | 4003.4        | -0.2    | 4003.4        | -0.2    | 3748.3        | -6.6    |
|                      | 80         | 5418.9             | 5403.0        | -0.3    | 5403.0        | -0.3    | 5403.0        | -0.3    | 4959.4        | -       |
|                      | 81         | -                  | -             | -       | -             | -       | 5614.7        | -       | -             | -       |
|                      | 85         | -                  | -             | -       | -             | -       | -             | -       | 5824.1        | -       |

The moment resistance specified by LRFD Specifications was found to be equal to the resistance provided by the Standard specifications for most of the cases with 6 ft. girder spacing. For other girder spacing, the moment resistance specified by LRFD Specifications is less conservative as compared to the Standard Specifications. The conservatism was found to be increasing with increase in span length and decreasing with the increase in girder spacing and skew angle. For skew angle of 60° the moment resistance was found to be smaller as compared to the Standard Specifications.

The difference between the moment resistance capacities predicted by Standard and LRFD Specifications is found to be in the range of -9.4 percent to 6 percent for 6 ft., -21 percent to 15 percent for 8 ft. and -16 percent to 7 percent for 8.67 ft. girder spacing. The comparison of the moment capacities is presented in Table 6.18.

## **6.6 IMPACT OF LRFD SPECIFICATIONS ON CAMBER**

The camber was calculated using the Hyperbolic Functions method proposed by Sinno et al. (1968). This method is used by TxDOT for the evaluation of camber. As the camber is evaluated using the same methodology for both the specifications, a small difference between the cambers is observed. The results for the camber are summarized in Table 6.19. The cambers for LRFD designs were generally smaller as compared to those for standard designs. This decrease is due to larger prestress losses, which decreases the prestressing force in the girder. The maximum difference in the camber is 22 percent for 6 ft. girder spacing, 13 percent for 8 ft. girder spacing and 11 percent for 8.67 ft. girder spacing.

**Table 6.19. Comparison of Camber for Type C Girder (Strand Dia. = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. Camber (ft.) | LRFD         |         |              |         |              |         |              |         |
|----------------------|------------|-------------------|--------------|---------|--------------|---------|--------------|---------|--------------|---------|
|                      |            |                   | skew = 0°    |         | skew = 15°   |         | skew = 30°   |         | skew = 60°   |         |
|                      |            |                   | Camber (ft.) | Diff. % | Camber (ft.) | Diff. % | Camber (ft.) | Diff. % | Camber (ft.) | Diff. % |
| 6                    | 40         | 0.041             | 0.032        | -22.3   | 0.032        | -22.3   | 0.032        | -22.3   | 0.032        | -22.3   |
|                      | 50         | 0.052             | 0.052        | 0.2     | 0.052        | 0.2     | 0.052        | 0.2     | 0.052        | 0.2     |
|                      | 60         | 0.090             | 0.087        | -2.9    | 0.087        | -2.9    | 0.089        | -1.1    | 0.089        | -1.1    |
|                      | 70         | 0.156             | 0.153        | -1.4    | 0.153        | -1.4    | 0.153        | -1.4    | 0.129        | -17.3   |
|                      | 80         | 0.220             | 0.241        | 9.4     | 0.241        | 9.4     | 0.241        | 9.4     | 0.216        | -1.8    |
|                      | 90         | 0.312             | 0.327        | 4.5     | 0.327        | 4.5     | 0.331        | 5.9     | 0.310        | -0.7    |
|                      | 95         | -                 | 0.372        | -       | 0.372        | -       | 0.355        | -       | -            | -       |
|                      | 96         | 0.366             | -            | -       | -            | -       | -            | -       | -            | -       |
| 8                    | 40         | 0.049             | 0.041        | -16.3   | 0.041        | -16.3   | 0.041        | -16.3   | 0.032        | -35.3   |
|                      | 50         | 0.065             | 0.077        | 18.1    | 0.077        | 18.1    | 0.064        | -0.9    | 0.051        | -20.8   |
|                      | 60         | 0.124             | 0.123        | -1.2    | 0.123        | -1.2    | 0.123        | -1.2    | 0.108        | -13.4   |
|                      | 70         | 0.211             | 0.207        | -1.6    | 0.207        | -1.6    | 0.207        | -1.6    | 0.192        | -9.0    |
|                      | 80         | 0.313             | 0.302        | -3.6    | 0.302        | -3.6    | 0.308        | -1.8    | 0.293        | -6.4    |
|                      | 83         | 0.342             | 0.347        | 1.3     | 0.347        | 1.3     | 0.336        | -1.9    | -            | -       |
|                      | 87         | -                 | -            | -       | -            | -       | -            | -       | 0.348        | -       |
| 8.67                 | 40         | 0.049             | 0.041        | -16.2   | 0.041        | -16.2   | 0.041        | -16.2   | 0.032        | -35.3   |
|                      | 50         | 0.076             | 0.077        | 0.3     | 0.077        | 0.3     | 0.077        | 0.3     | 0.064        | -15.9   |
|                      | 60         | 0.124             | 0.122        | -1.8    | 0.122        | -1.8    | 0.123        | -1.2    | 0.106        | -14.6   |
|                      | 70         | 0.229             | 0.225        | -1.5    | 0.225        | -1.5    | 0.225        | -1.5    | 0.207        | -9.4    |
|                      | 80         | 0.337             | 0.330        | -1.9    | 0.330        | -1.9    | 0.330        | -1.8    | 0.308        | -       |
|                      | 81         | -                 | -            | -       | -            | -       | 0.345        | -       | -            | -       |
|                      | 85         | -                 | -            | -       | -            | -       | -            | -       | 0.365        | -       |

## 6.7 IMPACT OF AASHTO LRFD ON SHEAR DESIGN

The LRFD Specifications employs a different methodology for the transverse shear design as compared to the Standard Specifications. This difference in the methodology impacted the transverse shear design significantly. The transverse shear reinforcement area was found to be increasing significantly as compared to the Standard Specifications. The interface shear reinforcement area was found to be increasing significantly for the LRFD Specifications. These changes in the transverse and interface shear reinforcements and results are addressed in Section 7.

## **7. SHEAR DESIGN**

### **7.1 INTRODUCTION**

The transverse shear design and the interface shear design are the two areas where significant differences between Standard and LRFD designs were observed in the parametric study results. These differences are caused due to a significant increase in the shear force specified by LRFD Specifications. The increase in concrete strength and the new approach for transverse shear design in the LRFD Specifications also affect the transverse shear design. The Standard Specifications uses a constant angle truss analogy for its shear provisions whereas, the LRFD specifications uses Modified Compression Field Theory (MCFT) based on variable angle truss analogy for shear provisions. The interface shear design in LRFD Specifications is based on the pure shear friction model whereas Standard Specifications uses empirical formulas for interface shear design provisions. This section includes the results for the interface and transverse shear design from the parametric study. Guidelines are also included on each of the two areas to help in the transition from Standard to LRFD design.

### **7.2 IMPACT OF LRFD SPECIFICATIONS ON TRANSVERSE SHEAR DESIGN**

#### **7.2.1 General**

This section includes a brief background of MCFT and the results for the transverse shear design for Standard and LRFD designs. Based on the results recommendations and guidelines are provided for implementation of LRFD Specifications in bridge design.

### 7.2.2. Modified Compression Field Theory (MCFT)

Modified Compression Field Theory is one of the methods for sectional analysis based on equilibrium, compatibility and stress-strain relationships. MCFT is a rational method, based on variable angle truss analogy (as compared to the constant  $45^\circ$  truss analogy used by traditional theories). MCFT provides a unified method for design, applicable to both the prestressed and nonprestressed concrete members. MCFT accounts for the tension in the longitudinal reinforcement due to shear and the stress transfer across the cracks. MCFT takes into account the shear stress and strain conditions at the section. The shear strength of concrete is determined using a factor  $\beta$ , which indicates the ability of diagonally cracked concrete to transfer tension. The angle of inclination of diagonal compressive stress,  $\theta$  is used to determine the critical section for shear. If  $\theta$  is taken as  $45^\circ$  and  $\beta$  as 2, this theory yields same results as  $45^\circ$  truss analogy, used in the Standard specifications. The use of MCFT in the LRFD Specifications results in a shear design procedure which is entirely different from the Standard Specifications. The LRFD Specifications has provided an extensive background of the mechanics and development of MCFT model, which can be very useful for bridge engineers to understand and implement the MCFT in shear designs.

The transverse shear design using MCFT results in a very complex design process. The MCFT is not suitable for routine bridge design. A research is being carried out at University of Illinois, to develop simplified shear design procedures for the use of bridge engineers. These formulas can be helpful for TxDOT engineers, if their applicability to the typical Texas bridges is verified. A similar research is being carried out at the Purdue University to establish simplified design expressions for shear design.







## **7.3 IMPACT OF LRFD SPECIFICATIONS ON INTERFACE SHEAR DESIGN**

### **7.3.1 General**

This section includes the results for the interface shear design for Standard and LRFD designs. The LRFD Specifications provide the cohesion and friction factors for two cases one, when the interface is roughened and another when the interface is not roughened. Both these cases were evaluated and the results are summarized. The proposed provisions to be included in LRFD Specifications are also investigated.

### **7.3.2 Impact on AASHTO Type IV Bridge Design**

The interface shear reinforcement area is found to be increasing significantly for LRFD designs for the case when the interface is roughened. A comparison of the interface shear reinforcement area for Standard and LRFD designs is presented in Table 7.3. The interface shear reinforcement area is found to be increasing in the range of 0 percent to 145 percent for 6 ft. girder spacing. This difference is found to be in the range of 16 percent to 218 percent for 8 ft. girder spacing and 40 percent to 192 percent for 8.67 ft. girder spacing.

**Table 7.3. Comparison of Interface Shear Reinforcement Area ( $A_{vh}$ ) with Roughened Interface (Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. $A_{vh}$ (in. <sup>2</sup> /ft.) | LRFD                             |         |                                  |         |                                  |         |                                  |         |
|----------------------|------------|---------------------------------------|----------------------------------|---------|----------------------------------|---------|----------------------------------|---------|----------------------------------|---------|
|                      |            |                                       | skew = 0°                        |         | skew = 15°                       |         | skew = 30°                       |         | skew = 60°                       |         |
|                      |            |                                       | $A_{vh}$ (in. <sup>2</sup> /ft.) | Diff. % | $A_{vh}$ (in. <sup>2</sup> /ft.) | Diff. % | $A_{vh}$ (in. <sup>2</sup> /ft.) | Diff. % | $A_{vh}$ (in. <sup>2</sup> /ft.) | Diff. % |
| 6                    | 90         | 0.20                                  | 0.20                             | 0.0     | 0.20                             | 0.0     | 0.20                             | 0.0     | 0.24                             | 21.4    |
|                      | 100        | 0.20                                  | 0.20                             | 0.0     | 0.20                             | 0.0     | 0.20                             | 0.0     | 0.29                             | 46.9    |
|                      | 110        | 0.20                                  | 0.20                             | 0.0     | 0.22                             | 9.2     | 0.25                             | 23.9    | 0.35                             | 76.2    |
|                      | 120        | 0.20                                  | 0.26                             | 29.2    | 0.29                             | 43.6    | 0.31                             | 57.0    | 0.43                             | 114.6   |
|                      | 130        | 0.20                                  | 0.31                             | 55.3    | 0.34                             | 71.2    | 0.37                             | 84.8    | 0.49                             | 145.6   |
|                      | 133        | -                                     | 0.35                             | -       | 0.38                             | -       | 0.41                             | -       | -                                | -       |
|                      | 135        | -                                     | -                                | -       | -                                | -       | -                                | -       | 0.55                             | -       |
|                      | 136        | 0.20                                  | -                                | -       | -                                | -       | -                                | -       | -                                | -       |
| 8                    | 90         | 0.20                                  | 0.23                             | 16.0    | 0.26                             | 28.9    | 0.29                             | 43.6    | 0.39                             | 94.7    |
|                      | 100        | 0.20                                  | 0.28                             | 41.8    | 0.31                             | 56.1    | 0.34                             | 70.2    | 0.46                             | 128.6   |
|                      | 110        | 0.20                                  | 0.34                             | 69.3    | 0.37                             | 85.1    | 0.40                             | 101.3   | 0.53                             | 166.4   |
|                      | 120        | 0.20                                  | 0.45                             | 124.0   | 0.48                             | 142.3   | 0.51                             | 155.5   | 0.64                             | 217.9   |
|                      | 124        | 0.20                                  | -                                | -       | -                                | -       | -                                | -       | -                                | -       |
|                      | 125        | -                                     | -                                | -       | -                                | -       | -                                | -       | 0.74                             | -       |
| 8.67                 | 90         | 0.20                                  | 0.27                             | 33.8    | 0.30                             | 47.5    | 0.33                             | 63.2    | 0.44                             | 118.1   |
|                      | 100        | 0.20                                  | 0.32                             | 61.0    | 0.35                             | 76.1    | 0.39                             | 93.5    | 0.51                             | 153.7   |
|                      | 110        | 0.20                                  | 0.38                             | 90.7    | 0.42                             | 107.5   | 0.45                             | 124.1   | 0.58                             | 192.4   |
|                      | 116        | -                                     | 0.45                             | -       | 0.49                             | -       | -                                | -       | -                                | -       |
|                      | 117        | -                                     | -                                | -       | -                                | -       | 0.54                             | -       | -                                | -       |
|                      | 119        | 0.20                                  | -                                | -       | -                                | -       | -                                | -       | -                                | -       |
|                      | 120        | -                                     | -                                | -       | -                                | -       | -                                | -       | 0.71                             | -       |
|                      | 121        | -                                     | -                                | -       | -                                | -       | -                                | -       | 0.73                             | -       |

The interface shear reinforcement area is found to be increasing significantly for LRFD designs for the case of unroughened interface. A comparison of the interface shear reinforcement area for Standard and LRFD designs is presented in Table 7.4. The interface shear reinforcement area is found to be increasing in the range of 75 percent to 392 percent for 6 ft. girder spacing. This difference is found to be in the range of 180 percent to 513 percent for 8 ft. girder spacing and 200 percent to 470 percent for 8.67 ft. girder spacing.

**Table 7.4. Comparison of Interface Shear Reinforcement Area ( $A_{vh}$ ) without Roughened Interface (Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. $A_{vh}$ (in. <sup>2</sup> /ft.) | LRFD                             |         |                                  |         |                                  |         |                                  |         |
|----------------------|------------|---------------------------------------|----------------------------------|---------|----------------------------------|---------|----------------------------------|---------|----------------------------------|---------|
|                      |            |                                       | skew = 0°                        |         | skew = 15°                       |         | skew = 30°                       |         | skew = 60°                       |         |
|                      |            |                                       | $A_{vh}$ (in. <sup>2</sup> /ft.) | Diff. % | $A_{vh}$ (in. <sup>2</sup> /ft.) | Diff. % | $A_{vh}$ (in. <sup>2</sup> /ft.) | Diff. % | $A_{vh}$ (in. <sup>2</sup> /ft.) | Diff. % |
| 6                    | 90         | 0.20                                  | 0.35                             | 75.2    | 0.39                             | 92.7    | 0.42                             | 112.5   | 0.57                             | 185.6   |
|                      | 100        | 0.20                                  | 0.42                             | 110.1   | 0.46                             | 129.4   | 0.50                             | 149.0   | 0.66                             | 228.2   |
|                      | 110        | 0.20                                  | 0.49                             | 144.1   | 0.53                             | 165.3   | 0.58                             | 189.8   | 0.75                             | 276.9   |
|                      | 120        | 0.20                                  | 0.60                             | 198.7   | 0.65                             | 222.7   | 0.69                             | 245.0   | 0.88                             | 341.0   |
|                      | 130        | 0.20                                  | 0.68                             | 242.2   | 0.74                             | 268.6   | 0.78                             | 291.3   | 0.99                             | 392.7   |
|                      | 133        | -                                     | 0.75                             | -       | 0.80                             | -       | 0.86                             | -       | -                                | -       |
|                      | 135        | -                                     | -                                | -       | -                                | -       | -                                | -       | 1.09                             | -       |
|                      | 136        | 0.20                                  | -                                | -       | -                                | -       | -                                | -       | -                                | -       |
| 8                    | 90         | 0.20                                  | 0.55                             | 176.7   | 0.60                             | 198.1   | 0.65                             | 222.7   | 0.82                             | 307.9   |
|                      | 100        | 0.20                                  | 0.64                             | 219.7   | 0.69                             | 243.5   | 0.73                             | 267.1   | 0.93                             | 364.4   |
|                      | 110        | 0.20                                  | 0.73                             | 265.5   | 0.78                             | 291.8   | 0.84                             | 318.8   | 1.05                             | 427.3   |
|                      | 120        | 0.20                                  | 0.91                             | 356.6   | 0.97                             | 387.2   | 1.02                             | 409.2   | 1.23                             | 513.1   |
|                      | 124        | 0.20                                  | -                                | -       | -                                | -       | -                                | -       | -                                | -       |
|                      | 125        | -                                     | -                                | -       | -                                | -       | -                                | -       | 1.39                             | -       |
| 8.67                 | 90         | 0.20                                  | 0.61                             | 206.3   | 0.66                             | 229.2   | 0.71                             | 255.4   | 0.89                             | 346.8   |
|                      | 100        | 0.20                                  | 0.70                             | 251.6   | 0.75                             | 276.8   | 0.81                             | 305.9   | 1.01                             | 406.2   |
|                      | 110        | 0.20                                  | 0.80                             | 301.2   | 0.86                             | 329.2   | 0.91                             | 356.9   | 1.14                             | 470.7   |
|                      | 116        | -                                     | 0.92                             | -       | 0.98                             | -       | -                                | -       | -                                | -       |
|                      | 117        | -                                     | -                                | -       | -                                | -       | 1.06                             | -       | -                                | -       |
|                      | 119        | 0.20                                  | -                                | -       | -                                | -       | -                                | -       | -                                | -       |
|                      | 120        | -                                     | -                                | -       | -                                | -       | -                                | -       | 1.35                             | -       |
|                      | 121        | -                                     | -                                | -       | -                                | -       | -                                | -       | 1.39                             | -       |

### 7.3.3 Impact on Type C Bridge Design

The interface shear reinforcement area is found to be increasing significantly for LRFD designs for the case when the interface is roughened. A comparison of the interface shear reinforcement area for Standard and LRFD designs is presented in Table 7.5. The interface shear reinforcement area is found to be increasing in the range of 2 percent to 411 percent for 6 ft. girder spacing. This difference is found to be in the range

of 67 percent to 330 percent for 8 ft. girder spacing and 87 percent to 284 percent for 8.67 ft. girder spacing.

**Table 7.5. Comparison of Interface Shear Reinforcement Area ( $A_{vh}$ ) for Type C Girder with Roughened Interface (Strand Diameter = 0.5 in.)**

| Girder Spacing (ft.) | Span (ft.) | Std. $A_{vh}$ (in. <sup>2</sup> /ft.) | LRFD                             |         |                                  |         |                                  |         |                                  |         |
|----------------------|------------|---------------------------------------|----------------------------------|---------|----------------------------------|---------|----------------------------------|---------|----------------------------------|---------|
|                      |            |                                       | skew = 0°                        |         | skew = 15°                       |         | skew = 30°                       |         | skew = 60°                       |         |
|                      |            |                                       | $A_{vh}$ (in. <sup>2</sup> /ft.) | Diff. % | $A_{vh}$ (in. <sup>2</sup> /ft.) | Diff. % | $A_{vh}$ (in. <sup>2</sup> /ft.) | Diff. % | $A_{vh}$ (in. <sup>2</sup> /ft.) | Diff. % |
| 6                    | 40         | 0.14                                  | 0.14                             | 1.4     | 0.16                             | 14.3    | 0.18                             | 29.1    | 0.26                             | 84.5    |
|                      | 50         | 0.14                                  | 0.21                             | 49.7    | 0.23                             | 65.6    | 0.26                             | 83.9    | 0.35                             | 152.3   |
|                      | 60         | 0.14                                  | 0.27                             | 95.3    | 0.30                             | 114.2   | 0.33                             | 135.9   | 0.44                             | 217.2   |
|                      | 70         | 0.14                                  | 0.33                             | 136.9   | 0.36                             | 158.6   | 0.40                             | 183.7   | 0.52                             | 273.3   |
|                      | 80         | 0.14                                  | 0.40                             | 183.6   | 0.43                             | 208.5   | 0.47                             | 237.2   | 0.61                             | 339.0   |
|                      | 90         | 0.14                                  | 0.47                             | 234.8   | 0.51                             | 263.1   | 0.55                             | 295.8   | 0.72                             | 411.6   |
|                      | 95         | -                                     | 0.53                             | -       | 0.58                             | -       | 0.61                             | -       | -                                | -       |
|                      | 96         | 0.17                                  | -                                | -       | -                                | -       | -                                | -       | -                                | -       |
|                      | 98         | -                                     | -                                | -       | -                                | -       | -                                | -       | 0.83                             | -       |
| 8                    | 40         | 0.14                                  | 0.23                             | 67.2    | 0.26                             | 82.7    | 0.28                             | 100.8   | 0.37                             | 166.4   |
|                      | 50         | 0.14                                  | 0.33                             | 133.4   | 0.35                             | 153.0   | 0.38                             | 171.2   | 0.49                             | 248.4   |
|                      | 60         | 0.14                                  | 0.40                             | 186.0   | 0.43                             | 209.1   | 0.47                             | 235.7   | 0.60                             | 330.9   |
|                      | 70         | 0.20                                  | 0.48                             | 145.5   | 0.52                             | 164.7   | 0.56                             | 186.9   | 0.71                             | 265.5   |
|                      | 80         | 0.26                                  | 0.56                             | 117.6   | 0.61                             | 134.3   | 0.66                             | 153.5   | 0.83                             | 221.7   |
|                      | 83         | 0.29                                  | 0.60                             | 110.3   | 0.65                             | 126.2   | 0.69                             | 141.4   | -                                | -       |
|                      | 87         | -                                     | -                                | -       | -                                | -       | -                                | -       | 0.92                             | -       |
| 8.67                 | 40         | 0.14                                  | 0.26                             | 86.8    | 0.28                             | 103.2   | 0.31                             | 122.3   | 0.41                             | 191.8   |
|                      | 50         | 0.14                                  | 0.36                             | 156.2   | 0.39                             | 176.8   | 0.42                             | 200.7   | 0.54                             | 284.5   |
|                      | 60         | 0.18                                  | 0.44                             | 143.8   | 0.47                             | 162.8   | 0.51                             | 184.8   | 0.65                             | 263.6   |
|                      | 70         | 0.25                                  | 0.52                             | 109.0   | 0.56                             | 124.9   | 0.61                             | 143.2   | 0.77                             | 208.7   |
|                      | 80         | 0.32                                  | 0.62                             | 92.3    | 0.67                             | 106.6   | 0.72                             | 123.1   | 0.90                             | -       |
|                      | 81         | -                                     | -                                | -       | -                                | -       | 0.73                             | -       | -                                | -       |
|                      | 85         | -                                     | -                                | -       | -                                | -       | -                                | -       | 0.98                             | -       |

The interface shear reinforcement area is found to be increasing significantly for LRFD designs for the case of unroughened interface. A comparison of the interface shear reinforcement area for Standard and LRFD designs is presented in Table 7.6. The interface shear reinforcement area is found to be increasing in the range of 152 percent to 836 percent for 6 ft. girder spacing. This difference is found to be in the range of 263

percent to 700 percent for 8 ft. girder spacing and 294 percent to 624 percent for 8.67 ft. girder spacing.

**Table 7.6. Comparison of Interface Shear Reinforcement Area ( $A_{vh}$ ) for Type C Girder without Roughened Interface (Strand Diameter = 0.5 in.)**

[illegible]



It was observed that the proposed provisions significantly reduce the interface shear reinforcement area requirement. The interface shear reinforcement requirement from the proposed provisions is same as that required by the Standard Specifications for all the cases for Type IV girders and most of the cases for Type C girders. The variation in the interface shear requirement for Type C girders is small.

**Table 7.8. Comparison of Interface Shear Reinforcement Area ( $A_{vh}$ ) for Type C Girder for Proposed Provisions (Strand Diameter = 0.5 in.)**

[illegible]

## **8. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS**

### **8.1 SUMMARY**

The main objective of this study was to develop guidelines to help TxDOT adopt and implement the AASHTO LRFD Bridge Design Specifications with a focus on AASHTO Type IV and Type C prestressed concrete bridge girders. Several tasks were performed to accomplish this objective. First, a review of the available literature on the development of AASHTO LRFD Specifications was carried out. A brief summary of the findings was documented. Second, detailed design examples were generated as a reference for bridge engineers to follow step by step designs based on the Standard and LRFD Specifications. Major differences in the designs using AASHTO Standard and LRFD Specifications were highlighted. Third, the simplification made by TxDOT in the bridge design by using the modular ratio between slab and girder concrete as unity was evaluated for its applicability when using the AASHTO LRFD Specifications. Fourth, a parametric study based on parameters representative of Texas bridges was conducted to investigate the impact of the AASHTO LRFD Specifications on the design as compared to the AASHTO Standard Specifications. The impact of the LRFD Specifications on service design, ultimate flexural design, shear design, and camber was evaluated. Fifth, based on the results from parametric study, areas where major differences were occurring in the design were identified as the transverse and interface shear design. Additional information and recommendations for these critical design issues were provided to help in the implementation of the LRFD Specifications in bridge designs by bridge engineers.

The following significant changes were found between the Standard and LRFD Specifications.

1. The live load model has changed significantly. The standard specifications use the greater of HS-20 truck or lane loading for live load. The LRFD Specifications use a HL-93 model, which is greater of the combination of HS-20 truck and lane loading and tandem and lane loading.



2. The dynamic load (impact) factor has changed. The impact factor is specified as 33 percent of live load in the LRFD Specifications which is significantly greater than the impact factors obtained in the Standard design.
3. The load combinations provided by the LRFD Specifications are different from those specified by the Standard Specifications. A new load combination, Service III, is specified by the LRFD Specifications for the tensile stress check in prestressed concrete members. A factor of 0.8 is applied to the live load moments in this load combination. This decreases the design tensile stress in the girder, neutralizing the effect of increased live load moments. The load factors for the ultimate flexural design load combination, Strength I are less than the ones provided by the Standard Specifications.

## **8.2 CONCLUSIONS**

Major conclusions from this study are provided in this section.

### **8.2.1 Parametric Study**

The following conclusions were derived from the parametric study for AASHTO Type IV and Type C girders.

1. Typically the combination of truck and lane load governs the LRFD designs. The tandem load and the lane load specified by the LRFD Specifications are different from those specified by the Standard Specifications.
2. The HL-93 load yields significantly larger moments and shears as compared to the HS-20 truck load.
3. The live load moment and shear distribution factors (DFs) have changed significantly. The DFs provided by the LRFD specifications are restrictive and can be used only if the specified limits are satisfied. The live load moment DFs specified by the LRFD Specifications are smaller as compared to the Standard DFs.

4. The live load shear DFs specified by LRFD are typically larger as compared to the Standard DFs. The skew reduction factors are applied to the live load moment DFs and a skew correction is applied to the shear DFs.
5. The distributed live load moments for LRFD designs are greater than the Standard designs. The distributed shear increased significantly as compared to the Standard Specifications.
6. The initial and final prestress losses in the LRFD designs are slightly greater than the ones obtained in the Standard designs.
7. The required number of strands in the LRFD design is slightly larger as compared to the Standard design. This increase is due to the increase in prestress losses and live load moments.
8. The required concrete strengths at release and at service in the LRFD designs are slightly greater than the ones obtained in the Standard design. This increase is due to the increase in the number of strands, which increase the stresses in the girder, requiring larger concrete strengths.
9. The overall impact of LRFD Specifications on the service load design of the prestressed concrete bridges is very small. The LRFD designs are generally slightly conservative as compared to the Standard designs.
10. The effect of the LRFD Specifications on the maximum span length is negligible. Slightly smaller span lengths are possible using the LRFD Specifications for skew angles less than 30 degrees. However, slightly larger span lengths are possible when the skew angle of 60 degrees is used. This is due to the significant decrease in live load moments for skew angles greater than 30 degrees.
11. The effect of LRFD Specifications on the ultimate flexural design is negligible. A small variation is observed in the design flexural moment and the flexural moment resistance as compared to the Standard designs.

12. A significant change was observed in the transverse shear design. The area of transverse reinforcement increased up to 300 percent in few cases. This increase in the transverse shear reinforcement is caused due to significant increase in the live load shear and a different methodology for transverse shear used in the LRFD Specifications. The LRFD Specifications uses Modified Compression Field Theory (MCFT) for its shear provisions whereas the Standard Specifications uses constant angle truss analogy.
13. The interface shear reinforcement area increased significantly for LRFD designs. The increase is up to 300% in some cases and 200% in most cases. This increase is caused due to conservative cohesion and friction factors specified by LRFD Specifications, based on pure friction model. However, the interface shear provisions proposed to be included in the LRFD Specifications in 2007 yields the shear reinforcement area which is comparable to the Standard Specifications.

#### **8.2.2. Effect of Modular Ratio**

The following are the findings from the evaluation of the TxDOT practice of not updating the modular ratio between slab and girder concrete in the design process

1. The impact of this practice is negligible in most of the cases. In few cases however a small difference was found, where the design using TxDOT methodology is on the unconservative side.
2. The LRFD live load moment and shear DFs were found to be decreasing by a small amount and consequently the live load moments and shears decreased slightly when the modular ratio was updated.
3. The service load design parameters, required number of strands, required concrete strengths at service and at release were found to be increasing by a small amount in few cases. There was no effect of updating the modular ratio for most of the cases.

4. The interface shear design is not affected by the process of updating the modular ratio. However the transverse shear reinforcement area requirement decreased for a few cases due to increase in concrete strengths, which subsequently increases the shear capacity of concrete.
5. The camber is found to be decreasing for a few cases, due to increase in the concrete strength which subsequently increases the elastic modulus of the concrete.

### **8.3 RECOMMENDATIONS FOR FUTURE RESEARCH**

1. Presently the LRFD Specifications are calibrated for the ultimate flexural design. The service load design needs to be calibrated to obtain a true reliability based specifications.
2. The live load DFs specified by LRFD Specifications are very restrictive. More research is needed to expand the approximate DFs specified by LRFD Specifications, to a wide range of bridge configurations.
3. The transverse shear design using MCFT is very complex design process. The MCFT is not suitable for routine bridge design. Simplified formulas could be helpful for the bridge engineers for transverse shear design. Research is being carried on at University of Illinois to arrive at simplified shear formulas. However, research is needed to find the applicability of any simplified formulas for typical Texas bridges.
4. The interface shear provisions proposed to be included in the LRFD Specifications in 2007 can be used after its inclusion into LRFD. However, for the bridge designs until then interface shear design criteria is needed to be evolved based on past experience and research studies on typical Texas bridges.

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**APPENDIX A**

**DETAILED DESIGN EXAMPLES FOR INTERIOR AASHTO TYPE**

**IV PRESTRESSED CONCRETE BRIDGE GIRDER**

## **Appendix A.1**

### **Detailed Example for Interior AASHTO Type IV Prestressed Concrete Bridge Girder Design using AASHTO Standard Specifications**

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## A.1 Interior AASHTO Type IV Prestressed Concrete Bridge Girder Design using AASHTO Standard Specifications

### A.1.1 INTRODUCTION

Following is a detailed example showing sample calculations for the design of a typical interior AASHTO Type IV prestressed concrete girder supporting a single span bridge. The design is based on the *AASHTO Standard Specifications for Highway Bridges, 17<sup>th</sup> Edition, 2002* (AASHTO 2002). The guidelines provided by the TxDOT Bridge Design Manual (TxDOT 2001) are considered in the design. The number of strands and concrete strength at release and at service are optimized using the TxDOT methodology.

### A.1.2 DESIGN PARAMETERS

The bridge considered for this design example has a span length of 110 ft. (c/c pier distance), a total width of 46 ft. and total roadway width of 44 ft. The bridge superstructure consists of six AASHTO Type IV girders spaced 8 ft. center-to-center, designed to act compositely with an 8 in. thick cast-in-place (CIP) concrete deck. The wearing surface thickness is 1.5 in., which includes the thickness of any future wearing surface. T501 type rails are considered in the design. The design live load is taken as either HS 20-44 truck or HS 20-44 lane load, whichever produces larger effects. A relative humidity (RH) of 60 percent is considered in the design. The bridge cross-section is shown in Figure A.1.2.1.

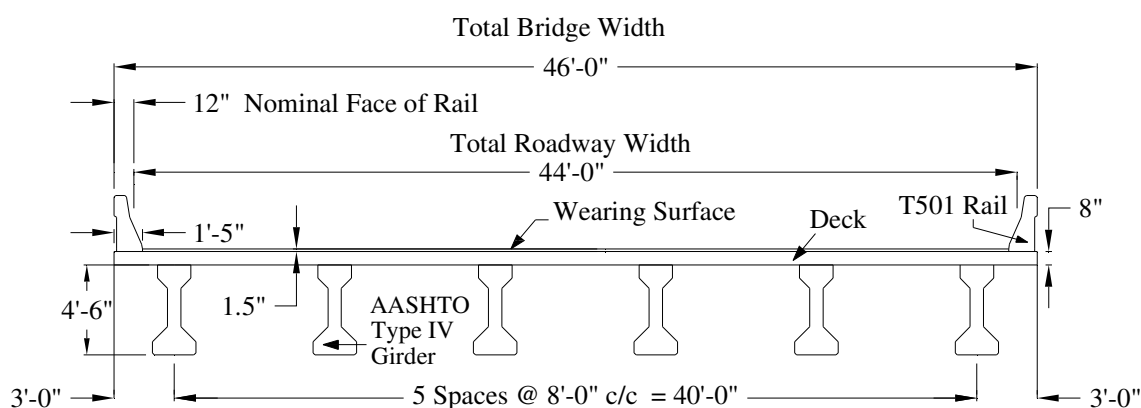


Figure A.1.2.1. Bridge Cross-Section Details.

The following calculations for design span length and the overall girder length are based on Figure A.1.2.2.

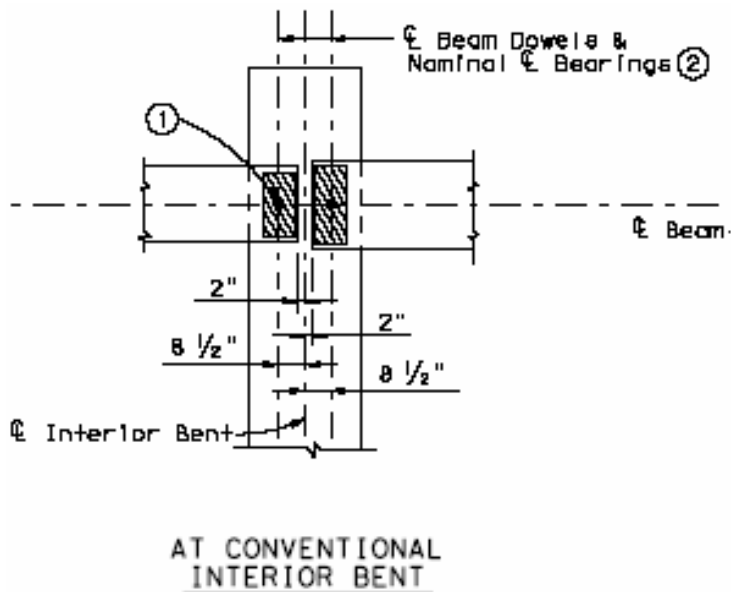


Figure A.1.2.2. Girder End Details  
(TxDOT Standard Drawing 2001).

Span Length (c/c piers) = 110'-0"

From Figure A.1.2.2

Overall girder length = 110'-0" - 2(2") = 109'-8" = 109.67 ft.

Design Span = 110'-0" - 2(8.5") = 108'-7" = 108.583 ft. (c/c of bearing)

### A.1.3 MATERIAL PROPERTIES

Cast-in-place slab:

Thickness,  $t_s = 8.0$  in.

Concrete strength at 28 days,  $f'_c = 4000$  psi

Thickness of asphalt-wearing surface (including any future wearing surface),  $t_w = 1.5$  in.

Unit weight of concrete,  $w_c = 150$  pcf

Precast girders: AASHTO Type IV

Concrete strength at release,  $f'_{ci} = 4000$  psi (This value is taken as an initial estimate and will be finalized based on optimum design.)

Concrete strength at 28 days,  $f'_c = 5000$  psi (This value is taken as initial estimate and will be finalized based on optimum design.)

Concrete unit weight,  $w_c = 150$  pcf

Pretensioning Strands: 0.5 in. diameter, seven wire low-relaxation  
Area of one strand = 0.153 in.<sup>2</sup>

Ultimate stress,  $f'_s = 270,000$  psi

Yield strength,  $f_y^* = 0.9 f'_s = 243,000$  psi [STD Art. 9.1.2]

Initial pretensioning,  $f_{si} = 0.75 f'_s$  [STD Art. 9.15.1]  
= 202,500 psi

Modulus of Elasticity,  $E_s = 28,000$  ksi [STD Art. 9.16.2.1.2]

Nonprestressed reinforcement: Yield strength,  $f_y = 60,000$  psi

Unit weight of asphalt-wearing surface = 140 pcf  
[TxDOT recommendation]

T501 type barrier weight = 326 plf /side

**A.1.4**  
**CROSS-SECTION**  
**PROPERTIES FOR A**  
**TYPICAL INTERIOR**  
**GIRDER**  
**A.1.4.1**  
**Non-Composite**  
**Section**

The section properties of an AASHTO Type IV girder as described in the TxDOT Bridge Design Manual (TxDOT 2001) are provided in Table A.1.4.1. The section geometry and strand pattern is shown in Figure A.1.4.1.

*Table A.1.4.1. Section Properties of AASHTO Type IV Girder  
[Adapted from TxDOT Bridge Design Manual (TxDOT 2001)].*

| $y_t$ | $y_b$ | Area             | $I$              | Wt./lf |
|-------|-------|------------------|------------------|--------|
| in.   | in.   | in. <sup>2</sup> | in. <sup>4</sup> | lbs    |
| 29.25 | 24.75 | 788.4            | 260,403          | 821    |

where

$I$  = Moment of inertia about the centroid of the non-composite precast girder, in.<sup>4</sup>

$y_b$  = Distance from centroid to the extreme bottom fiber of the non-composite precast girder, in.

$y_t$  = Distance from centroid to the extreme top fiber of the non-composite precast girder, in.

$S_b$  = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder, in.<sup>3</sup>  
 $= I/y_b = 260,403/24.75 = 10,521.33 \text{ in.}^3$

$S_t$  = Section modulus referenced to the extreme top fiber of the non-composite precast girder, in.<sup>3</sup>  
 $= I/y_t = 260,403/29.25 = 8902.67 \text{ in.}^3$

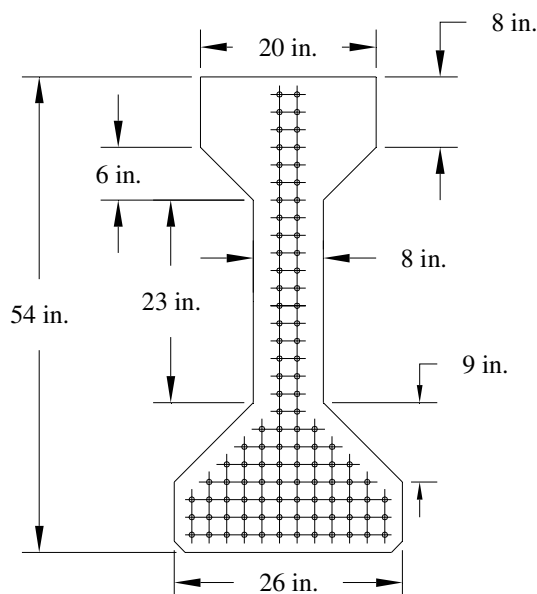


Figure A.1.4.1. Section Geometry and Strand Pattern for AASHTO Type IV Girder [Adapted from TxDOT Bridge Design Manual (TxDOT 2001)].

#### A.1.4.2

##### Composite Section

##### A.1.4.2.1

##### Effective Web Width

[STD Art. 9.8.3]

Effective web width of the precast girder is lesser of:

[STD Art. 9.8.3.1]

$$b_e = 6 \times (\text{flange thickness on either side of the web}) + \text{web} + \text{fillets} \\ = 6(8 + 8) + 8 + 2(6) = 116 \text{ in.}$$

$$\text{or, } b_e = \text{Total top flange width} = 20 \text{ in.} \quad (\text{controls})$$

Effective web width,  $b_e = 20 \text{ in.}$

### A.1.4.2.2 Effective Flange Width

The effective flange width is lesser of: [STD Art. 9.8.3.2]

$$\frac{1}{4} \text{ span length of girder: } \frac{108.583(12 \text{ in./ft.})}{4} = 325.75 \text{ in.}$$

$$6 \times (\text{effective slab thickness on each side of the effective web width}) + \text{effective web width: } 6(8 + 8) + 20 = 116 \text{ in.}$$

One-half the clear distance on each side of the effective web width + effective web width: For interior girders this is equivalent to the center-to-center distance between the adjacent girders.

$$8(12 \text{ in./ft.}) + 20 \text{ in.} = 96 \text{ in.} \quad (\text{controls})$$

Effective flange width = 96 in.

### A.1.4.2.3 Modular Ratio between Slab and Girder Concrete

Following the TxDOT Design Manual (TxDOT 2001) recommendation (pg. 7-85), the modular ratio between the slab and the girder concrete is taken as 1. This assumption is used for service load design calculations. For flexural strength limit design, shear design, and deflection calculations, the actual modular ratio based on optimized concrete strengths is used. The composite section is shown in Figure A.1.4.2 and the composite section properties are presented in Table A.1.4.2.

$$n = \left( \frac{E_c \text{ for slab}}{E_c \text{ for girder}} \right) = 1$$

where  $n$  is the modular ratio between slab and girder concrete, and  $E_c$  is the elastic modulus of concrete.

### A.1.4.2.4 Transformed Section Properties

$$\begin{aligned} \text{Transformed flange width} &= n \times (\text{effective flange width}) \\ &= (1)(96) = 96 \text{ in.} \end{aligned}$$

$$\begin{aligned} \text{Transformed Flange Area} &= n \times (\text{effective flange width})(t_s) \\ &= (1)(96)(8) = 768 \text{ in.}^2 \end{aligned}$$

Table A.1.4.2. Properties of Composite Section.

|          | Transformed Area<br>$A \text{ (in.}^2\text{)}$ | $y_b$<br>in. | $Ay_b$<br>in. <sup>3</sup> | $A(y_{bc} - y_b)^2$ | $I$<br>in. <sup>4</sup> | $I + A(y_{bc} - y_b)^2$<br>in. <sup>4</sup> |
|----------|--|--------------|----------------------------|---------------------|-------------------------|---|
| Girder   | 788.4  | 24.75        | 19,512.9                   | 212,231.53          | 260,403.0               | 472,634.5                                   |
| Slab     | 768.0  | 58.00        | 44,544.0                   | 217,868.93          | 4096.0                  | 221,964.9                                   |
| $\Sigma$ | 1556.4   |              | 64,056.9                   |                     |                         | 694,599.5                                   |

$$A_c = \text{Total area of composite section} = 1556.4 \text{ in.}^2$$

$$h_c = \text{Total height of composite section} = 54 \text{ in.} + 8 \text{ in.} = 62 \text{ in.}$$

$$I_c = \text{Moment of inertia about the centroid of the composite section} = 694,599.5 \text{ in.}^4$$

$$y_{bc} = \text{Distance from the centroid of the composite section to extreme bottom fiber of the precast girder, in.} \\ = 64,056.9/1,556.4 = 41.157 \text{ in.}$$

$$y_{tg} = \text{Distance from the centroid of the composite section to extreme top fiber of the precast girder, in.} \\ = 54 - 41.157 = 12.843 \text{ in.}$$

$$y_{tc} = \text{Distance from the centroid of the composite section to extreme top fiber of the slab, in.} \\ = 62 - 41.157 = 20.843 \text{ in.}$$

$$S_{bc} = \text{Section modulus of composite section referenced to the extreme bottom fiber of the precast girder, in.}^3 \\ = I_c/y_{bc} = 694,599.5/41.157 = 16,876.83 \text{ in.}^3$$

$$S_{tg} = \text{Section modulus of composite section referenced to the top fiber of the precast girder, in.}^3 \\ = I_c/y_{tg} = 694,599.5/12.843 = 54,083.9 \text{ in.}^3$$

$$S_{tc} = \text{Section modulus of composite section referenced to the top fiber of the slab, in.}^3 \\ = I_c/y_{tc} = 694,599.5/20.843 = 33,325.31 \text{ in.}^3$$

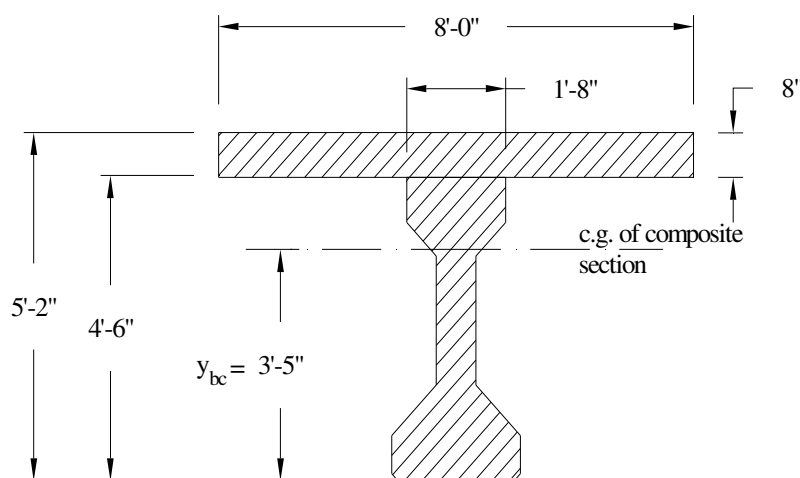


Figure A.1.4.2. Composite Section.



### **A.1.5 SHEAR FORCES AND BENDING MOMENTS**

The self-weight of the girder and the weight of the slab act on the non-composite simple span structure, while the weight of the barriers, future wearing surface, and live load including impact load act on the composite simple span structure.

#### **A.1.5.1 Shear Forces and Bending Moments due to Dead Loads**

##### **A.1.5.1.1 Dead Loads**

Dead loads acting on the non-composite structure:

Self-weight of the girder = 0.821 kips/ft.  
[TxDOT Bridge Design Manual (TxDOT 2001)]

Weight of cast-in-place deck on each interior girder

$$= (0.150 \text{ kcf}) \left( \frac{8 \text{ in.}}{12 \text{ in./ft.}} \right) (8 \text{ ft.}) = 0.800 \text{ kips/ft.}$$

Total dead load on non-composite section  
= 0.821 + 0.800 = 1.621 kips/ft.

##### **A.1.5.1.2 Superimposed Dead Loads**

The dead loads placed on the composite structure are distributed equally among all the girders.  
[STD Art. 3.23.2.3.1.1 & TxDOT Bridge Design Manual pg. 6-13]

Weight of T501 rails or barriers on each girder

$$= 2 \left( \frac{326 \text{ plf} / 1000}{6 \text{ girders}} \right) = 0.109 \text{ kips/ft./girder}$$

Weight of 1.5 in. wearing surface

$$= (0.140 \text{ kcf}) \left( \frac{1.5 \text{ in.}}{12 \text{ in./ft.}} \right) = 0.0175 \text{ ksf. This is applied over the}$$

entire clear roadway width of 44'-0".

$$\text{Weight of wearing surface on each girder} = \frac{(0.0175 \text{ ksf})(44.0 \text{ ft.})}{6 \text{ girders}}$$

$$= 0.128 \text{ kips/ft./girder}$$

Total superimposed dead load = 0.109 + 0.128 = 0.237 kips/ft.

##### **A.1.5.1.3 Shear Forces and Bending Moments**

Shear forces and bending moments for the girder due to dead loads, superimposed dead loads at every tenth of the design span, and at critical sections (hold-down point or harp point and critical section

for shear) are provided in this section. The bending moment ( $M$ ) and shear force ( $V$ ) due to uniform dead loads and uniform superimposed dead loads at any section at a distance  $x$  from the centerline of bearing are calculated using the following formulas, where the uniform dead load is denoted as  $w$ .

$$M = 0.5wx(L - x)$$

$$V = w(0.5L - x)$$

The critical section for shear is located at a distance  $h_c/2$  from the face of the support. However, as the support dimensions are not specified in this study, the critical section is measured from the centerline of bearing. This yields a conservative estimate of the design shear force.

Distance of critical section for shear from centerline of bearing  
 $= 62/2 = 31 \text{ in.} = 2.583 \text{ ft.}$

As per the recommendations of the TxDOT Bridge Design Manual (Chap. 7, Sec. 21), the distance of the hold-down ( $HD$ ) point from the centerline of bearing is taken as the lesser of:

$[0.5 \times (\text{span length}) - (\text{span length}/20)]$  or  $[0.5 \times (\text{span length}) - 5 \text{ ft.}]$

$$\frac{108.583}{2} - \frac{108.583}{20} = 48.862 \text{ ft. or } \frac{108.583}{2} - 5 = 49.29 \text{ ft.}$$

$HD = 48.862 \text{ ft.}$

The shear forces and bending moments due to dead loads and superimposed dead loads are shown in Table A.1.5.1.

*Table A.1.5.1. Shear Forces and Bending Moments due to Dead and Superimposed Dead Loads.*

| Distance from Bearing Centerline<br>$x$<br>ft. | Section<br>$x/L$  | Dead Load     |         |             |         | Superimposed Dead Loads |        | Total Dead Load |         |
|--|-------------------|---------------|---------|-------------|---------|-------------------------|--------|-----------------|---------|
|  |                   | Girder Weight |         | Slab Weight |         | Shear                   | Moment | Shear           | Moment  |
|  |                   | Shear         | Moment  | Shear       | Moment  |                         |        |                 |         |
|  |                   | kips          | k-ft.   | kips        | k-ft.   | kips                    | k-ft.  | kips            | k-ft.   |
| 0.000  | 0.000             | 44.57         | 0.00    | 43.43       | 0.00    | 12.87                   | 0.00   | 100.87          | 0.00    |
| 2.583  | 0.024 ( $h_c/2$ ) | 42.45         | 112.39  | 41.37       | 109.52  | 12.25                   | 32.45  | 96.07           | 254.36  |
| 10.858   | 0.100             | 35.66         | 435.59  | 34.75       | 424.45  | 10.29                   | 125.74 | 80.70           | 985.78  |
| 21.717   | 0.200             | 26.74         | 774.38  | 26.06       | 754.58  | 7.72                    | 223.54 | 60.52           | 1752.51 |
| 32.575   | 0.300             | 17.83         | 1016.38 | 17.37       | 990.38  | 5.15                    | 293.40 | 40.35           | 2300.16 |
| 43.433   | 0.400             | 8.91          | 1161.58 | 8.69        | 1131.87 | 2.57                    | 335.32 | 20.17           | 2628.76 |
| 48.862   | 0.450 ( $HD$ )    | 4.46          | 1197.87 | 4.34        | 1167.24 | 1.29                    | 345.79 | 10.09           | 2710.90 |
| 54.292   | 0.500             | 0.00          | 1209.98 | 0.00        | 1179.03 | 0.00                    | 349.29 | 0.00            | 2738.29 |

**A.1.5.2**  
**Shear Forces and**  
**Bending Moments**  
**due to Live Load**  
**A.1.5.2.1**  
**Live Load**

The AASHTO Standard Specifications require the live load to be taken as either HS 20-44 standard truck loading, lane loading, or tandem loading-whichever yields the largest moments and shears. For spans longer than 40 ft., tandem loading does not govern; thus only HS 20-44 truck loading and lane loading are investigated here. [STD Art. 3.7.1.1]

The unfactored bending moments ( $M$ ) and shear forces ( $V$ ) due to HS 20-44 truck loading on a per-lane-basis are calculated using the following formulas given in the *PCI Design Manual* (PCI 2003).

Maximum bending moment due to HS 20-44 truck load

For  $x/L = 0 - 0.333$

$$M = \frac{72(x)[(L - x) - 9.33]}{L}$$

For  $x/L = 0.333 - 0.5$

$$M = \frac{72(x)[(L - x) - 4.67]}{L} - 112$$

Maximum shear force due to HS 20-44 truck load

For  $x/L = 0 - 0.5$

$$V = \frac{72[(L - x) - 9.33]}{L}$$

The bending moments and shear forces due to HS 20-44 lane load are calculated using the following formulas given in the *PCI Design Manual* (PCI 2003).

Maximum bending moment due to HS 20-44 lane load

$$M = \frac{P(x)(L - x)}{L} + 0.5(w)(x)(L - x)$$

Maximum shear force due to HS 20-44 lane load

$$V = \frac{Q(L - x)}{L} + (w)\left(\frac{L}{2} - x\right)$$

where

$x$  = Distance from the centerline of bearing to the section at which bending moment or shear force is calculated, ft.

$L$  = Design span length = 108.583 ft.

$P$  = Concentrated load for moment = 18 kips

$Q$  = Concentrated load for shear = 26 kips

$w$  = Uniform load per linear foot of lane = 0.64 klf

Shear force and bending moment due to live load including impact loading is distributed to individual girders by multiplying the distribution factor and the impact factor as follows.

Bending moment due to live load including impact load  
 $M_{LL+I}$  = (live load bending moment per lane) ( $DF$ ) ( $1+I$ )

Shear force due to live load including impact load  
 $V_{LL+I}$  = (live load shear force per lane) ( $DF$ ) ( $1+I$ )

where  $DF$  is the live load distribution factor, and  $I$  is the live load impact factor.

#### **A.1.5.2.2 Live Load Distribution Factor for a Typical Interior Girder**

The live load distribution factor for moment, for a precast prestressed concrete interior girder, is given by the following expression:

$$DF_{mom} = \frac{S}{5.5} = \frac{8.0}{5.5} = 1.4545 \text{ wheels/girder} \quad [\text{STD Table 3.23.1}]$$

where

$S$  = Average spacing between girders in feet = 8 ft.

The live load distribution factor for an individual girder is obtained as  $DF = DF_{mom}/2 = 0.727$  lanes/girder.

For simplicity of calculation and because there is no significant difference, the distribution factor for moment is used also for shear as recommended by the TxDOT Bridge Design Manual (Chap. 6, Sec. 3, TxDOT 2001).

#### **A.1.5.2.3 Live Load Impact**

[STD Art. 3.8]

The live load impact factor is given by the following expression:

$$I = \frac{50}{L + 125} \quad [\text{STD Eq. 3-1}]$$

where

$I$  = Impact fraction to a maximum of 30 percent

$L$  = Design span length in feet = 108.583 ft. [STD Art. 3.8.2.2]

$$I = \frac{50}{108.583 + 125} = 0.214$$

The impact factor for shear varies along the span according to the location of the truck, but the impact factor computed above is also used for shear for simplicity as recommended by the TxDOT Bridge Design Manual (TxDOT 2001).

The distributed shear forces and bending moments due to live load are provided in Table A.1.5.2.

*Table A.1.5.2. Distributed Shear Forces and Bending Moments due to Live Load.*

| Distance from Bearing Centerline<br>$x$ | Section<br>$x/L$  | HS 20-44 Truck Loading (controls) |         |                    |         | HS 20-44 Lane Loading |         |                    |         |
|---|-------------------|-----------------------------------|---------|--------------------|---------|-----------------------|---------|--------------------|---------|
|   |                   | Live Load                         |         | Live Load + Impact |         | Live Load             |         | Live Load + Impact |         |
|   |                   | Shear                             | Moment  | Shear              | Moment  | Shear                 | Moment  | Shear              | Moment  |
|   |                   | kips                              | k-ft.   | kips               | k-ft.   | kips                  | k-ft.   | kips               | k-ft.   |
| 0.000                                   | 0.000             | 65.81                             | 0.00    | 58.11              | 0.00    | 60.75                 | 0.00    | 53.64              | 0.00    |
| 2.583                                   | 0.024 ( $h_c/2$ ) | 64.10                             | 165.57  | 56.60              | 146.19  | 58.47                 | 133.00  | 51.63              | 117.44  |
| 10.858                                  | 0.100             | 58.61                             | 636.44  | 51.75              | 561.95  | 51.20                 | 515.46  | 45.20              | 455.13  |
| 21.717                                  | 0.200             | 51.41                             | 1116.52 | 45.40              | 985.84  | 41.65                 | 916.38  | 36.77              | 809.12  |
| 32.575                                  | 0.300             | 44.21                             | 1440.25 | 39.04              | 1271.67 | 32.10                 | 1202.75 | 28.34              | 1061.97 |
| 43.433                                  | 0.400             | 37.01                             | 1629.82 | 32.68              | 1439.05 | 22.55                 | 1374.57 | 19.91              | 1213.68 |
| 48.862                                  | 0.450 ( $HD$ )    | 33.41                             | 1671.64 | 29.50              | 1475.97 | 17.77                 | 1417.52 | 15.69              | 1251.60 |
| 54.292                                  | 0.500             | 29.81                             | 1674.37 | 26.32              | 1478.39 | 13.00                 | 1431.84 | 11.48              | 1264.25 |

### **A.1.5.3** **Load Combination**

[STD Art. 3.22]

This design example considers only the dead and vehicular live loads. The wind load and the earthquake load are not included in the design, which is typical for the design of bridges in Texas. The general expression for group loading combinations for service load design (SLD) and load factor design (LFD) considering dead and live loads is given as:

$$\text{Group } (N) = \gamma[\beta_D \times D + \beta_L \times (L + I)]$$

where:

$N$  = Group number

$\gamma$  = Load factor given by STD Table 3.22.1.A.

$\beta$  = Coefficient given by STD Table 3.22.1.A.

$D$  = Dead load

$L$  = Live load

$I$  = Live load impact

Various group combinations provided by STD Table. 3.22.1.A are investigated, and the following group combinations are found to be applicable in the present case.

For service load design

Group I: This group combination is used for design of members for 100 percent basic unit stress. [STD Table 3.22.1A]

$$\gamma = 1.0$$

$$\beta_D = 1.0$$

$$\beta_L = 1.0$$

$$\text{Group (I)} = 1.0 \times (D) + 1.0 \times (L+I)$$

For load factor design

Group I: This load combination is the general load combination for load factor design relating to the normal vehicular use of the bridge. [STD Table 3.22.1A]

$$\gamma = 1.3$$

$$\beta_D = 1.0 \text{ for flexural and tension members.}$$

$$\beta_L = 1.67$$

$$\text{Group (I)} = 1.3[1.0 \times (D) + 1.67 \times (L+I)]$$

### **A.1.6**

#### **ESTIMATION OF REQUIRED PRESTRESS**

#### **A.1.6.1**

##### **Service Load Stresses at Midspan**

The required number of strands is usually governed by concrete tensile stress at the bottom fiber of the girder at midspan section. The service load combination, Group I, is used to evaluate the bottom fiber stresses at the midspan section. The calculation for compressive stress in the top fiber of the girder at midspan section under Group I service load combination is shown in the following section.

Tensile stress at bottom fiber of the girder at midspan due to applied loads

$$f_b = \frac{M_g + M_s}{S_b} + \frac{M_{SDL} + M_{LL+I}}{S_{bc}}$$

Compressive stress at top fiber of the girder at midspan due to applied loads

$$f_t = \frac{M_g + M_s}{S_t} + \frac{M_{SDL} + M_{LL+I}}{S_{tg}}$$

where:

$f_b$  = Concrete stress at the bottom fiber of the girder at the midspan section, ksi

$f_t$  = Concrete stress at the top fiber of the girder at the midspan section, ksi

$M_g$  = Moment due to girder self-weight at the midspan section of the girder = 1209.98 k-ft.

$M_s$  = Moment due to slab weight at the midspan section of the girder = 1179.03 k-ft.

$M_{SDL}$  = Moment due to superimposed dead loads at the midspan section of the girder = 349.29 k-ft.

$M_{LL+I}$  = Moment due to live load including impact load at the midspan section of the girder = 1478.39 k-ft.

$S_b$  = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder = 10,521.33 in.<sup>3</sup>

$S_t$  = Section modulus referenced to the extreme top fiber of the non-composite precast girder = 8902.67 in.<sup>3</sup>

$S_{bc}$  = Section modulus of composite section referenced to the extreme bottom fiber of the precast girder = 16,876.83 in.<sup>3</sup>

$S_{tg}$  = Section modulus of composite section referenced to the top fiber of the precast girder = 54,083.9 in.<sup>3</sup>

Substituting the bending moments and section modulus values, the stresses at bottom fiber ( $f_b$ ) and top fiber ( $f_t$ ) of the girder at the midspan section are:

$$f_b = \frac{(1209.98 + 1179.03)(12 \text{ in./ft.})}{10,521.33} + \frac{(349.29 + 1478.39)(12 \text{ in./ft.})}{16,876.83}$$

$$= 4.024 \text{ ksi}$$

$$f_t = \frac{(1209.98 + 1179.03)(12 \text{ in./ft.})}{8902.67} + \frac{(349.29 + 1478.39)(12 \text{ in./ft.})}{54,083.9}$$

$$= 3.626 \text{ ksi}$$

The stresses at the top and bottom fibers of the girder at the hold-down point, midspan and top fiber of the slab are calculated in a similar fashion as shown above and summarized in Table A.1.6.1.

Table A.1.6.1. Summary of Stresses due to Applied Loads.

| Load                   | Stresses in Girder                |                    |                   |                    | Stresses in Slab at Midspan |
|------------------------|-----------------------------------|--------------------|-------------------|--------------------|-----------------------------|
|                        | Stress at Hold-Down ( <i>HD</i> ) |                    | Stress at Midspan |                    |                             |
|                        | Top Fiber (psi)                   | Bottom Fiber (psi) | Top Fiber (psi)   | Bottom Fiber (psi) | Top Fiber (psi)             |
| Girder Self-weight     | 1614.63                           | -1366.22           | 1630.94           | -1380.03           | -                           |
| Slab Weight            | 1573.33                           | -1331.28           | 1589.22           | -1344.73           | -                           |
| Superimposed Dead Load | 76.72                             | -245.87            | 77.50             | 248.35             | 125.77                      |
| Total Dead Load        | 3264.68                           | -2943.37           | 3297.66           | -2973.10           | 125.77                      |
| Live Load              | 327.49                            | -1049.47           | 328.02            | -1051.19           | 532.35                      |
| Total Load             | 3592.17                           | -3992.84           | 3625.68           | -4024.29           | 658.12                      |

(Negative values indicate tensile stresses)

### A.1.6.2 Allowable Stress Limit

At service load conditions, the allowable tensile stress for members with bonded prestressed reinforcement is:

$$F_b = 6\sqrt{f'_c} = 6\sqrt{5000} \left( \frac{1}{1000} \right) = 0.4242 \text{ ksi} \quad [\text{STD Art. 9.15.2.2}]$$

### A.1.6.3 Required Number of Strands

Required precompressive stress in the bottom fiber after losses:

Bottom tensile stress – allowable tensile stress at final =  $f_b - F_b$

$$f_{b-reqd.} = 4.024 - 0.4242 = 3.60 \text{ ksi}$$

Assuming the eccentricity of the prestressing strands at midspan ( $e_c$ ) as the distance from the centroid of the girder to the bottom fiber of the girder (PSTRS 14 methodology, TxDOT 2001)

$$e_c = y_b = 24.75 \text{ in.}$$

Bottom fiber stress due to prestress after losses:

$$f_b = \frac{P_{se}}{A} + \frac{P_{se} e_c}{S_b}$$

where:

$P_{se}$  = Effective pretension force after all losses, kips

$A$  = Area of girder cross-section = 788.4 in.<sup>2</sup>

$S_b$  = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder = 10,521.33 in.<sup>3</sup>



Required pretension is calculated by substituting the corresponding values in the above equation as follows:

$$3.60 = \frac{P_{se}}{788.4} + \frac{P_{se}(24.75)}{10,521.33}$$

Solving for  $P_{se}$ ,  
 $P_{se} = 994.27$  kips

Assuming final losses = 20 percent of initial prestress,  $f_{si}$  (TxDOT 2001)

$$\text{Assumed final losses} = 0.2(202.5) = 40.5 \text{ ksi}$$

The prestress force per strand after losses  
 = (cross-sectional area of one strand) [ $f_{si}$  – losses]  
 =  $0.153(202.5 - 40.5) = 24.78$  kips

$$\text{Number of prestressing strands required} = 994.27/24.78 = 40.12$$

Try 42 – 0.5 in. diameter, 270 ksi low-relaxation strands as an initial estimate.

Strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2+4+6) + 6(8)}{42} = 20.18 \text{ in.}$$

Available prestressing force

$$P_{se} = 42(24.78) = 1040.76 \text{ kips}$$

Stress at bottom fiber of the girder at midspan due to prestressing, after losses

$$f_b = \frac{1040.76}{788.4} + \frac{1040.76(20.18)}{10,521.33}$$

$$= 1.320 + 1.996 = 3.316 \text{ ksi} < f_{b-reqd.} = 3.60 \text{ ksi}$$

Try 44 – 0.5 in. diameter, 270 ksi low-relaxation strands as an initial estimate.

Strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2+4+6) + 8(8)}{44} = 20.02 \text{ in.}$$

Available prestressing force

$$P_{se} = 44(24.78) = 1090.32 \text{ kips}$$

Stress at bottom fiber of the girder at midspan due to prestressing, after losses

$$f_b = \frac{1090.32}{788.4} + \frac{1090.32(20.02)}{10,521.33} = 1.383 + 2.074 = 3.457 \text{ ksi} < f_{b-reqd.} = 3.60 \text{ ksi}$$

Try 46 – 0.5 in. diameter, 270 ksi low-relaxation strands as an initial estimate

Effective strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2+4+6) + 10(8)}{46} = 19.88 \text{ in.}$$

Available prestressing force is:

$$P_{se} = 46(24.78) = 1139.88 \text{ kips}$$

Stress at bottom fiber of the girder at midspan due to prestressing, after losses

$$f_b = \frac{1139.88}{788.4} + \frac{1139.88(19.88)}{10,521.33} = 1.446 + 2.153 = 3.599 \text{ ksi} \sim f_{b-reqd.} = 3.601 \text{ ksi}$$

Therefore, 46 strands are used as a preliminary estimate for the number of strands. The strand arrangement is shown in Figure A.1.6.1.

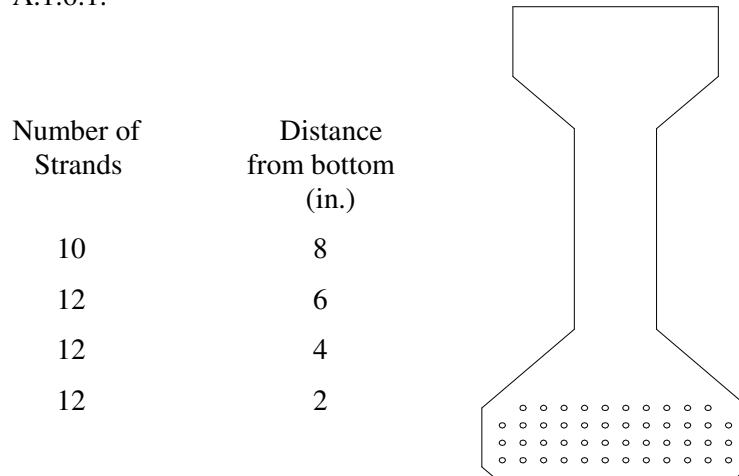


Figure A.1.6.1. Initial Strand Arrangement.

The distance from the centroid of the strands to the bottom fiber of the girder ( $y_{bs}$ ) is calculated as:

$$y_{bs} = y_b - e_c = 24.75 - 19.88 = 4.87 \text{ in.}$$

### **A.1.7 PRESTRESS LOSSES**

Total prestress losses =  $SH + ES + CR_C + CR_S$  [STD Art. 9.16.2]  
[STD Eq. 9-3]

where:

$SH$  = Loss of prestress due to concrete shrinkage, ksi

$ES$  = Loss of prestress due to elastic shortening, ksi

$CR_C$  = Loss of prestress due to creep of concrete, ksi

$CR_S$  = Loss of prestress due to relaxation of pretensioning steel, ksi

Number of strands = 46

A number of iterations based on TxDOT methodology (TxDOT 2001) will be performed to arrive at the optimum number of strands, required concrete strength at release ( $f'_{ci}$ ), and required concrete strength at service ( $f'_c$ ).

#### **A.1.7.1 Iteration 1 A.1.7.1.1 Concrete Shrinkage**

[STD Art. 9.16.2.1.1]  
For pretensioned members, the loss in prestress due to concrete shrinkage is given as:

$$SH = 17,000 - 150 RH \quad [STD Eq. 9-4]$$

where:

$RH$  is the relative humidity = 60 percent

$$SH = [17,000 - 150(60)] \frac{1}{1000} = 8.0 \text{ ksi}$$

#### **A.1.7.1.2 Elastic Shortening**

[STD Art. 9.16.2.1.2]  
For pretensioned members, the loss in prestress due to elastic shortening is given as:

$$ES = \frac{E_s}{E_{ci}} f_{cir} \quad [STD Eq. 9-6]$$

where:

$f_{cir}$  = Average concrete stress at the center of gravity of the pretensioning steel due to the pretensioning force and the dead load of girder immediately after transfer, ksi

$$= \frac{P_{si}}{A} + \frac{P_{si} e_c^2}{I} - \frac{(M_g)_c}{I}$$

$P_{si}$  = Pretension force after allowing for the initial losses, kips

As the initial losses are unknown at this point, 8 percent initial loss in prestress is assumed as a first estimate.

$$P_{si} = (\text{number of strands})(\text{area of each strand})[0.92(0.75 f'_s)] \\ = 46(0.153)(0.92)(0.75)(270) = 1311.18 \text{ kips}$$

$$M_g = \text{Moment due to girder self-weight at midspan, k-ft.} \\ = 1209.98 \text{ k-ft.}$$

$$e_c = \text{Eccentricity of the prestressing strands at the midspan} \\ = 19.88 \text{ in.}$$

$$f_{cir} = \frac{1311.18}{788.4} + \frac{1311.18(19.88)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.88)}{260,403} \\ = 1.663 + 1.990 - 1.108 = 2.545 \text{ ksi}$$

Initial estimate for concrete strength at release,  $f'_{ci} = 4000$  psi

Modulus of elasticity of girder concrete at release is given as:

$$E_{ci} = 33(w_c)^{3/2} \sqrt{f'_{ci}} \quad [\text{STD Eq. 9-8}] \\ = [33(150)^{3/2} \sqrt{4000}] \left( \frac{1}{1000} \right) = 3834.25 \text{ ksi}$$

Modulus of elasticity of prestressing steel,  $E_s = 28,000$  ksi

Prestress loss due to elastic shortening is:

$$ES = \left[ \frac{28,000}{3834.25} \right] (2.545) = 18.59 \text{ ksi}$$

### **A.1.7.1.3 Creep of Concrete**

[STD Art. 9.16.2.1.3]

The loss in prestress due to the creep of concrete is specified to be calculated using the following formula:

$$CR_C = 12f_{cir} - 7f_{cds} \quad [\text{STD Eq. 9-9}]$$

where:

$$f_{cds} = \text{Concrete stress at the center of gravity of the prestressing steel due to all dead loads except the dead load present at the time the prestressing force is applied, ksi} \\ = \frac{M_S e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

$M_{SDL}$  = Moment due to superimposed dead load at midspan section = 349.29 k-ft.

$M_S$  = Moment due to slab weight at midspan section = 1179.03 k-ft.

$y_{bc}$  = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder = 41.157 in.

$y_{bs}$  = Distance from center of gravity of the prestressing strands at midspan to the bottom fiber of the girder = 24.75 – 19.88 = 4.87 in.

$I$  = Moment of inertia of the non-composite section = 260,403 in.<sup>4</sup>

$I_c$  = Moment of inertia of composite section = 694,599.5 in.<sup>4</sup>

$$f_{cds} = \frac{1179.03(12 \text{ in./ft.})(19.88)}{260,403} + \frac{349.29(12 \text{ in./ft.})(41.157 - 4.87)}{694,599.5}$$

$$= 1.080 + 0.219 = 1.299 \text{ ksi}$$

Prestress loss due to creep of concrete is:

$$CR_C = 12(2.545) - 7(1.299) = 21.45 \text{ ksi}$$

#### **A.1.7.1.4 Relaxation of Prestressing Steel**

[STD Art. 9.16.2.1.4]

For pretensioned members with 270 ksi low-relaxation strands, the prestress loss due to relaxation of prestressing steel is calculated using the following formula.

$$CR_S = 5000 - 0.10ES - 0.05(SH + CR_C) \quad [\text{STD Eq. 9-10A}]$$

where the variables are the same as defined in Section A.1.7 expressed in psi units.

$$CR_S = [5000 - 0.10(18,590) - 0.05(8000 + 21,450)] \left( \frac{1}{1000} \right)$$

$$= 1.669 \text{ ksi}$$

The *PCI Design Manual* (PCI 2003) considers only the elastic shortening loss in the calculation of total initial prestress loss, whereas, the *TxDOT Bridge Design Manual* (pg. 7-85, TxDOT 2001) recommends that 50 percent of the final steel relaxation loss shall also be considered for calculation of total initial prestress loss given as:

$$[\text{elastic shortening loss} + 0.50(\text{total steel relaxation loss})]$$

Using the TxDOT Bridge Design Manual (TxDOT 2001) recommendations, the initial prestress loss is calculated as follows.

$$\begin{aligned}\text{Initial prestress loss} &= \frac{(ES + \frac{1}{2}CR_s)100}{0.75f'_s} \\ &= \frac{[18.59 + 0.5(1.669)]100}{0.75(270)} = 9.59\% > 8\% \text{ (assumed value of} \\ &\text{initial prestress loss)}\end{aligned}$$

Therefore, another trial is required assuming 9.59 percent initial prestress loss.

The change in initial prestress loss will not affect the prestress loss due to concrete shrinkage. Therefore, the next trials will involve updating the losses due to elastic shortening, steel relaxation, and creep of concrete.

Based on the initial prestress loss value of 9.59 percent, the pretension force after allowing for the initial losses is calculated as follows.

$$\begin{aligned}P_{si} &= (\text{number of strands})(\text{area of each strand})[0.904(0.75 f'_s)] \\ &= 46(0.153)(0.904)(0.75)(270) = 1288.38 \text{ kips}\end{aligned}$$

Loss in prestress due to elastic shortening is:

$$\begin{aligned}ES &= \frac{E_s}{E_{ci}} f_{cir} \\ f_{cir} &= \frac{P_{si}}{A} + \frac{P_{si}e_c^2}{I} - \frac{(M_g)e_c}{I} \\ f_{cir} &= \frac{1288.38}{788.4} + \frac{1288.38(19.88)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.88)}{260,403} \\ &= 1.634 + 1.955 - 1.108 = 2.481 \text{ ksi} \\ E_s &= 28,000 \text{ ksi} \\ E_{ci} &= 3834.25 \text{ ksi} \\ ES &= \left[ \frac{28,000}{3834.25} \right] (2.481) = 18.12 \text{ ksi}\end{aligned}$$

Loss in prestress due to creep of concrete

$$CR_C = 12f_{cir} - 7f_{cds}$$

The value of  $f_{cds}$  is independent of the initial prestressing force value and will be the same as calculated in Section A.1.7.1.3.

$$f_{cds} = 1.299 \text{ ksi}$$

$$CR_C = 12(2.481) - 7(1.299) = 20.68 \text{ ksi}$$

Loss in prestress due to relaxation of steel

$$\begin{aligned} CR_S &= 5000 - 0.10 ES - 0.05(SH + CR_C) \\ &= [5000 - 0.10(18,120) - 0.05(8000 + 20,680)] \left( \frac{1}{1000} \right) \\ &= 1.754 \text{ ksi} \end{aligned}$$

$$\begin{aligned} \text{Initial prestress loss} &= \frac{(ES + \frac{1}{2} CR_S) 100}{0.75 f'_s} \\ &= \frac{[18.12 + 0.5(1.754)] 100}{0.75(270)} = 9.38\% < 9.59\% \text{ (assumed value} \\ &\text{for initial prestress loss)} \end{aligned}$$

Therefore, another trial is required assuming 9.38 percent initial prestress loss.

Based on the initial prestress loss value of 9.38 percent, the pretension force after allowing for the initial losses is calculated as follows.

$$\begin{aligned} P_{si} &= (\text{number of strands})(\text{area of each strand})[0.906 (0.75 f'_s)] \\ &= 46(0.153)(0.906)(0.75)(270) = 1291.23 \text{ kips} \end{aligned}$$

Loss in prestress due to elastic shortening

$$\begin{aligned} ES &= \frac{E_s}{E_{ci}} f_{cir} \\ f_{cir} &= \frac{P_{si}}{A} + \frac{P_{si} e_c^2}{I} - \frac{(M_g) e_c}{I} \\ f_{cir} &= \frac{1291.23}{788.4} + \frac{1291.23(19.88)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.88)}{260,403} \\ &= 1.638 + 1.960 - 1.108 = 2.490 \text{ ksi} \\ E_s &= 28,000 \text{ ksi} \\ E_{ci} &= 3834.25 \text{ ksi} \\ ES &= \left[ \frac{28,000}{3834.25} \right] (2.490) = 18.18 \text{ ksi} \end{aligned}$$

Loss in prestress due to creep of concrete

$$CR_C = 12f_{cir} - 7f_{cds}$$

$$f_{cds} = 1.299 \text{ ksi}$$

$$CR_C = 12(2.490) - 7(1.299) = 20.79 \text{ ksi}$$

Loss in prestress due to relaxation of steel

$$CR_S = 5000 - 0.10 ES - 0.05(SH + CR_C)$$

$$= [5000 - 0.10(18,180) - 0.05(8000 + 20,790)] \left( \frac{1}{1000} \right)$$

$$= 1.743 \text{ ksi}$$

$$\text{Initial prestress loss} = \frac{(ES + \frac{1}{2} CR_S) 100}{0.75f'_s}$$

$$= \frac{[18.18 + 0.5(1.743)] 100}{0.75(270)} = 9.41\% \approx 9.38\% \text{ (assumed value}$$

of initial prestress loss)

#### **A.1.7.1.5 Total Losses at Transfer**

$$\text{Total prestress loss at transfer} = (ES + \frac{1}{2} CR_S)$$

$$= [18.18 + 0.5(1.743)] = 19.05 \text{ ksi}$$

$$\text{Effective initial prestress, } f_{si} = 202.5 - 19.05 = 183.45 \text{ ksi}$$

$P_{si}$  = Effective pretension after allowing for the initial prestress loss

$$= (\text{number of strands})(\text{area of strand})(f_{si})$$

$$= 46(0.153)(183.45) = 1291.12 \text{ kips}$$

#### **A.1.7.1.6 Total Losses at Service**

Loss in prestress due to concrete shrinkage,  $SH = 8.0 \text{ ksi}$

Loss in prestress due to elastic shortening,  $ES = 18.18 \text{ ksi}$

Loss in prestress due to creep of concrete,  $CR_C = 20.79 \text{ ksi}$

Loss in prestress due to steel relaxation,  $CR_S = 1.743 \text{ ksi}$

Total final loss in prestress =  $SH + ES + CR_C + CR_S$

$$= 8.0 + 18.18 + 20.79 + 1.743 = 48.71 \text{ ksi}$$

$$\text{or, } \frac{48.71(100)}{0.75(270)} = 24.06 \%$$

$$\text{Effective final prestress, } f_{se} = 0.75(270) - 48.71 = 153.79 \text{ ksi}$$



$$\begin{aligned}
 P_{se} &= \text{Effective pretension after allowing for the final prestress loss} \\
 &= (\text{number of strands})(\text{area of strand})(\text{effective final prestress}) \\
 &= 46(0.153)(153.79) = 1082.37 \text{ kips}
 \end{aligned}$$

#### **A.1.7.1.7** **Final Stresses at** **Midspan**

The number of strands is updated based on the final stress at the bottom fiber of the girder at the midspan section.

Final stress at the bottom fiber of the girder at the midspan section due to effective prestress,  $f_{bf}$ , is calculated as follows.

$$\begin{aligned}
 f_{bf} &= \frac{P_{se}}{A} + \frac{P_{se} e_c}{S_b} = \frac{1082.37}{788.4} + \frac{1082.37 (19.88)}{10,521.33} \\
 &= 1.373 + 2.045 = 3.418 \text{ ksi} < f_{b-reqd.} = 3.600 \text{ ksi} \quad (\text{N.G.}) \\
 & (f_{b-reqd.} \text{ calculations are presented in Section A.1.6.3})
 \end{aligned}$$

Try 48 – 0.5 in. diameter, 270 ksi low-relaxation strands

Eccentricity of prestressing strands at midspan

$$e_c = 24.75 - \frac{12(2+4+6) + 10(8) + 2(10)}{48} = 19.67 \text{ in.}$$

Effective pretension after allowing for the final prestress loss

$$P_{se} = 48(0.153)(153.79) = 1129.43 \text{ kips}$$

Final stress at the bottom fiber of the girder at midspan section due to effective prestress

$$\begin{aligned}
 f_{bf} &= \frac{1129.43}{788.4} + \frac{1129.43 (19.67)}{10,521.33} \\
 &= 1.432 + 2.11 = 3.542 \text{ ksi} < f_{b-reqd.} = 3.600 \text{ ksi} \quad (\text{N.G.})
 \end{aligned}$$

Try 50 – 0.5 in. diameter, 270 ksi low-relaxation strands

Eccentricity of prestressing strands at midspan

$$e_c = 24.75 - \frac{12(2+4+6) + 10(8) + 4(10)}{50} = 19.47 \text{ in.}$$

Effective pretension after allowing for the final prestress loss

$$P_{se} = 50(0.153)(153.79) = 1176.49 \text{ kips}$$

Final stress at the bottom fiber of the girder at midspan section due to effective prestress

$$f_{bf} = \frac{1176.49}{788.4} + \frac{1176.49 (19.47)}{10,521.33}$$

$$= 1.492 + 2.177 = 3.669 \text{ ksi} > f_{b-reqd.} = 3.600 \text{ ksi} \quad (\text{O.K.})$$

Therefore use 50 – 0.5 in. diameter, 270 ksi low-relaxation strands.

Concrete stress at the top fiber of the girder due to effective prestress and applied loads

$$f_{tf} = \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + f_i = \frac{1176.49}{788.4} - \frac{1176.49 (19.47)}{8902.67} + 3.626$$

$$= 1.492 - 2.573 + 3.626 = 2.545 \text{ ksi}$$

( $f_i$  calculations are presented in Section A.1.6.1)

#### **A.1.7.1.8** **Initial Stresses at** **Hold-Down Point**

The concrete strength at release,  $f'_{ci}$ , is updated based on the initial stress at the bottom fiber of the girder at the hold-down point.

Prestressing force after allowing for initial prestress loss

$$P_{si} = (\text{number of strands})(\text{area of strand})(\text{effective initial prestress})$$

$$= 50(0.153)(183.45) = 1403.39 \text{ kips}$$

(Effective initial prestress calculations are presented in Section A.1.7.1.5.)

Initial concrete stress at top fiber of the girder at the hold-down point due to self-weight of the girder and effective initial prestress

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si} e_c}{S_t} + \frac{M_g}{S_t}$$

where:

$$M_g = \text{Moment due to girder self-weight at hold-down point}$$

$$\text{based on overall girder length of } 109'-8''$$

$$= 0.5wx(L - x)$$

$$w = \text{Self-weight of the girder} = 0.821 \text{ kips/ft.}$$

$$L = \text{Overall girder length} = 109.67 \text{ ft.}$$

$$x = \text{Distance of hold-down point from the end of the girder}$$

$$= HD + (\text{distance from centerline of bearing to the girder end})$$

$HD =$  Hold-down point distance from centerline of the bearing  
 $= 48.862$  ft. (see Sec. A.1.5.1.3)

$$x = 48.862 + 0.542 = 49.404 \text{ ft.}$$

$$M_g = 0.5(0.821)(49.404)(109.67 - 49.404) = 1222.22 \text{ k-ft.}$$

$$\begin{aligned} f_{ti} &= \frac{1403.39}{788.4} - \frac{1403.39 (19.47)}{8902.67} + \frac{1222.22(12 \text{ in./ft.})}{8902.67} \\ &= 1.78 - 3.069 + 1.647 = 0.358 \text{ ksi} \end{aligned}$$

Initial concrete stress at bottom fiber of the girder at hold-down point due to self-weight of the girder and effective initial prestress

$$\begin{aligned} f_{bi} &= \frac{P_{si}}{A} + \frac{P_{si} e_c}{S_b} - \frac{M_g}{S_b} \\ f_{bi} &= \frac{1403.39}{788.4} + \frac{1403.39 (19.47)}{10,521.33} - \frac{1222.22(12 \text{ in./ft.})}{10,521.33} \\ &= 1.78 + 2.597 - 1.394 = 2.983 \text{ ksi} \end{aligned}$$

Compression stress limit for pretensioned members at transfer stage is  $0.6 f'_{ci}$  [STD Art. 9.15.2.1]

$$\text{Therefore, } f'_{ci \text{ -reqd.}} = \frac{2983}{0.6} = 4971.67 \text{ psi}$$

### **A.1.7.2 Iteration 2**

A second iteration is carried out to determine the prestress losses and subsequently estimate the required concrete strength at release and at service using the following parameters determined in the previous iteration.

Number of strands = 50

Concrete strength at release,  $f'_{ci} = 4971.67$  psi

#### **A.1.7.2.1 Concrete Shrinkage**

[STD Art. 9.16.2.1.1]

For pretensioned members, the loss in prestress due to concrete shrinkage is given as:

$$SH = 17,000 - 150 RH \quad [\text{STD Eq. 9-4}]$$

where  $RH$  is the relative humidity = 60 percent

$$SH = [17,000 - 150(60)] \frac{1}{1000} = 8.0 \text{ ksi}$$

### A.1.7.2.2 Elastic Shortening

[STD Art. 9.16.2.1.2]

For pretensioned members, the loss in prestress due to elastic shortening is given as:

$$ES = \frac{E_s}{E_{ci}} f_{cir} \quad [\text{STD Eq. 9-6}]$$

where:

$f_{cir}$  = Average concrete stress at the center of gravity of the pretensioning steel due to the pretensioning force and the dead load of girder immediately after transfer, ksi

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}e_c^2}{I} - \frac{(M_g)e_c}{I}$$

$P_{si}$  = Pretension force after allowing for the initial losses, kips

As the initial losses are dependent on the elastic shortening and steel relaxation loss, which are yet to be determined, the initial loss value of 9.41 percent obtained in the last trial of iteration 1 is taken as an initial estimate for initial loss in prestress.

$$\begin{aligned} P_{si} &= (\text{number of strands})(\text{area of strand})[0.9059(0.75 f'_s)] \\ &= 50(0.153)(0.9059)(0.75)(270) = 1403.35 \text{ kips} \end{aligned}$$

$$\begin{aligned} M_g &= \text{Moment due to girder self-weight at midspan, k-ft.} \\ &= 1209.98 \text{ k-ft.} \end{aligned}$$

$$\begin{aligned} e_c &= \text{Eccentricity of the prestressing strands at the midspan} \\ &= 19.47 \text{ in.} \end{aligned}$$

$$\begin{aligned} f_{cir} &= \frac{1403.35}{788.4} + \frac{1403.35(19.47)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.47)}{260,403} \\ &= 1.78 + 2.043 - 1.086 = 2.737 \text{ ksi} \end{aligned}$$

$$\text{Concrete strength at release, } f'_{ci} = 4971.67 \text{ psi}$$

Modulus of elasticity of girder concrete at release is given as:

$$\begin{aligned} E_{ci} &= 33(w_c)^{3/2} \sqrt{f'_{ci}} \quad [\text{STD Eq. 9-8}] \\ &= [33(150)^{3/2} \sqrt{4971.67}] \left( \frac{1}{1000} \right) = 4274.66 \text{ ksi} \end{aligned}$$

Modulus of elasticity of prestressing steel,  $E_s = 28,000$  ksi

Prestress loss due to elastic shortening is:

$$ES = \left[ \frac{28,000}{4274.66} \right] (2.737) = 17.93 \text{ ksi}$$

### A.1.7.2.3 Creep of Concrete

[STD Art. 9.16.2.1.3]

The loss in prestress due to creep of concrete is specified to be calculated using the following formula.

$$CR_C = 12f_{cir} - 7f_{cds} \quad [\text{STD Eq. 9-9}]$$

where:

$$f_{cds} = \frac{M_S e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

$M_{SDL}$  = Moment due to superimposed dead load at midspan section = 349.29 k-ft.

$M_S$  = Moment due to slab weight at midspan section  
= 1179.03 k-ft.

$y_{bc}$  = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder = 41.157 in.

$y_{bs}$  = Distance from center of gravity of the prestressing strands at midspan to the bottom fiber of the girder  
= 24.75 – 19.47 = 5.28 in.

$I$  = Moment of inertia of the non-composite section  
= 260,403 in.<sup>4</sup>

$I_c$  = Moment of inertia of composite section = 694,599.5 in.<sup>4</sup>

$$\begin{aligned} f_{cds} &= \frac{1179.03(12 \text{ in./ft.})(19.47)}{260,403} + \frac{(349.29)(12 \text{ in./ft.})(41.157 - 5.28)}{694,599.5} \\ &= 1.058 + 0.216 = 1.274 \text{ ksi} \end{aligned}$$

Prestress loss due to creep of concrete is

$$CR_C = 12(2.737) - 7(1.274) = 23.93 \text{ ksi}$$

**A.1.7.2.4**  
**Relaxation of**  
**Pretensioning Steel**

[STD Art. 9.16.2.1.4]

For pretensioned members with 270 ksi low-relaxation strands, prestress loss due to relaxation of prestressing steel is calculated using the following formula.

$$CR_S = 5000 - 0.10 ES - 0.05(SH + CR_C) \quad [\text{STD Eq. 9-10A}]$$

$$CR_S = [5000 - 0.10(17,930) - 0.05(8000 + 23,930)] \left( \frac{1}{1000} \right)$$

$$= 1.61 \text{ ksi}$$

$$\text{Initial prestress loss} = \frac{(ES + \frac{1}{2} CR_S) 100}{0.75 f'_s}$$

$$= \frac{[17.93 + 0.5(1.61)] 100}{0.75(270)} = 9.25\% < 9.41\% \text{ (assumed value of initial prestress loss)}$$

Therefore another trial is required assuming 9.25 percent initial prestress loss.

The change in initial prestress loss will not affect the prestress loss due to concrete shrinkage. Therefore, the next trial will involve updating the losses due to elastic shortening, steel relaxation, and creep of concrete.

Based on the initial prestress loss value of 9.25 percent, the pretension force after allowing for the initial losses is calculated as follows:

$$P_{si} = (\text{number of strands})(\text{area of each strand})[0.9075(0.75 f'_s)]$$

$$= 50(0.153)(0.9075)(0.75)(270) = 1405.83 \text{ kips}$$

Loss in prestress due to elastic shortening

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si} e_c^2}{I} - \frac{(M_g) e_c}{I}$$

$$= \frac{1405.83}{788.4} + \frac{1405.83(19.47)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.47)}{260,403}$$

$$= 1.783 + 2.046 - 1.086 = 2.743 \text{ ksi}$$

$$E_s = 28,000 \text{ ksi}$$

$$E_{ci} = 4274.66 \text{ ksi}$$

Prestress loss due to elastic shortening is:

$$ES = \left[ \frac{28,000}{4274.66} \right] (2.743) = 17.97 \text{ ksi}$$

Loss in prestress due to creep of concrete

$$CR_C = 12f_{cir} - 7f_{cds}$$

The value of  $f_{cds}$  is independent of the initial prestressing force value and will be the same as calculated in Section A.1.7.2.3.

$$f_{cds} = 1.274 \text{ ksi}$$

$$CR_C = 12(2.743) - 7(1.274) = 24.0 \text{ ksi}$$

Loss in prestress due to relaxation of steel

$$\begin{aligned} CR_S &= 5000 - 0.10 ES - 0.05(SH + CR_C) \\ &= [5000 - 0.10(17,970) - 0.05(8000 + 24,000)] \left( \frac{1}{1000} \right) \\ &= 1.603 \text{ ksi} \end{aligned}$$

$$\begin{aligned} \text{Initial prestress loss} &= \frac{(ES + \frac{1}{2} CR_S) 100}{0.75f'_s} \\ &= \frac{[17.97 + 0.5(1.603)] 100}{0.75(270)} = 9.27\% \approx 9.25\% \text{ (assumed value for initial prestress loss)} \end{aligned}$$

#### **A.1.7.2.5 Total Losses at Transfer**

$$\begin{aligned} \text{Total prestress loss at transfer} &= (ES + \frac{1}{2} CR_S) \\ &= [17.97 + 0.5(1.603)] = 18.77 \text{ ksi} \end{aligned}$$

$$\text{Effective initial prestress, } f_{si} = 202.5 - 18.77 = 183.73 \text{ ksi}$$

$$\begin{aligned} P_{si} &= \text{Effective pretension after allowing for the initial prestress loss} \\ &= (\text{number of strands})(\text{area of strand})(f_{si}) \\ &= 50(0.153)(183.73) = 1405.53 \text{ kips} \end{aligned}$$

#### **A.1.7.2.6 Total Losses at Service**

Loss in prestress due to concrete shrinkage,  $SH = 8.0 \text{ ksi}$

Loss in prestress due to elastic shortening,  $ES = 17.97 \text{ ksi}$

Loss in prestress due to creep of concrete,  $CR_C = 24.0 \text{ ksi}$

Loss in prestress due to steel relaxation,  $CR_S = 1.603 \text{ ksi}$

$$\begin{aligned}\text{Total final loss in prestress} &= SH + ES + CR_C + CR_S \\ &= 8.0 + 17.97 + 24.0 + 1.603 = 51.57 \text{ ksi}\end{aligned}$$

$$\text{or } \frac{51.57(100)}{0.75(270)} = 25.47 \%$$

$$\text{Effective final prestress, } f_{se} = 0.75(270) - 51.57 = 150.93 \text{ ksi}$$

$$\begin{aligned}P_{se} &= \text{Effective pretension after allowing for the final prestress loss} \\ &= (\text{number of strands})(\text{area of strand})(\text{effective final prestress}) \\ &= 50(0.153)(150.93) = 1154.61 \text{ kips}\end{aligned}$$

#### **A.1.7.2.7** **Final Stresses at** **Midspan**

Concrete stress at top fiber of the girder at the midspan section due to applied loads and effective prestress

$$\begin{aligned}f_{tf} &= \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + f_i = \frac{1154.61}{788.4} - \frac{1154.61 (19.47)}{8902.67} + 3.626 \\ &= 1.464 - 2.525 + 3.626 = 2.565 \text{ ksi}\end{aligned}$$

( $f_i$  calculations are presented in Section A.1.6.1.)

Compressive stress limit under service load combination is  $0.6 f'_c$   
[STD Art. 9.15.2.2]

$$f'_c \text{ -reqd.} = \frac{2565}{0.60} = 4275 \text{ psi}$$

Concrete stress at top fiber of the girder at midspan due to effective prestress + permanent dead loads

$$\begin{aligned}f_{tf} &= \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}} \\ &= \frac{1154.61}{788.4} - \frac{1154.61 (19.47)}{8902.67} + \frac{(1209.98 + 1179.03)(12 \text{ in./ft.})}{8902.67} \\ &\quad + \frac{349.29(12 \text{ in./ft.})}{54,083.9} \\ &= 1.464 - 2.525 + 3.22 + 0.077 = 2.236 \text{ ksi}\end{aligned}$$

Compressive stress limit for effective prestress + permanent dead loads =  $0.4 f'_c$   
[STD Art. 9.15.2.2]

$$f'_c \text{ -reqd.} = \frac{2236}{0.40} = 5590 \text{ psi} \quad (\text{controls})$$



Concrete stress at top fiber of the girder at midspan due to live load + 0.5(effective prestress + dead loads)

$$\begin{aligned}
 f_{tf} &= \frac{M_{LL+I}}{S_{tg}} + 0.5 \left( \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}} \right) \\
 &= \frac{1478.39(12 \text{ in./ft.})}{54083.9} + 0.5 \left\{ \frac{1154.61}{788.4} - \frac{1154.61(19.47)}{8902.67} + \right. \\
 &\quad \left. \frac{(1209.98 + 1179.03)(12 \text{ in./ft.})}{8902.67} + \frac{349.29(12 \text{ in./ft.})}{54,083.9} \right\} \\
 &= 0.328 + 0.5(1.464 - 2.525 + 3.22 + 0.077) = 1.446 \text{ ksi}
 \end{aligned}$$

Allowable limit for compressive stress due to live load + 0.5(effective prestress + dead loads) =  $0.4 f'_c$  [STD Art. 9.15.2.2]

$$f'_c \text{ -reqd.} = \frac{1446}{0.40} = 3615 \text{ psi}$$

Tensile stress at the bottom fiber of the girder at midspan due to service loads

$$\begin{aligned}
 f_{bf} &= \frac{P_{se}}{A} + \frac{P_{se} e_c}{S_b} - f_b \text{ (} f_b \text{ calculations are presented in Sec. A.1.6.1.)} \\
 &= \frac{1154.61}{788.4} + \frac{1154.61(19.47)}{10,521.33} - 4.024 \\
 &= 1.464 + 2.14 - 4.024 = -0.420 \text{ ksi (negative sign indicates} \\
 &\quad \text{tensile stress)}
 \end{aligned}$$

For members with bonded reinforcement allowable tension in the precompressed tensile zone =  $6\sqrt{f'_c}$  [STD Art. 9.15.2.2]

$$f'_c \text{ -reqd.} = \left( \frac{420}{6} \right)^2 = 4900 \text{ psi}$$

The concrete strength at service is updated based on the final stresses at the midspan section under different loading combinations. The required concrete strength at service is determined to be 5590 psi.

**A.1.7.2.8**  
**Initial Stresses at**  
**Hold-Down Point**

Prestressing force after allowing for initial prestress loss

$$P_{si} = (\text{number of strands})(\text{area of strand})(\text{effective initial prestress})$$

$$= 50(0.153)(183.73) = 1405.53 \text{ kips}$$

(Effective initial prestress calculations are presented in Section A.1.7.2.5.)

Initial concrete stress at top fiber of the girder at hold-down point due to self-weight of girder and effective initial prestress

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si} e_c}{S_t} + \frac{M_g}{S_t}$$

where:

$$M_g = \text{Moment due to girder self-weight at the hold-down point based on overall girder length of } 109'-8''$$

$$= 1222.22 \text{ k-ft. (see Section A.1.7.1.8)}$$

$$f_{ti} = \frac{1405.53}{788.4} - \frac{1405.53(19.47)}{8902.67} + \frac{1222.22(12 \text{ in./ft.})}{8902.67}$$

$$= 1.783 - 3.074 + 1.647 = 0.356 \text{ ksi}$$

Initial concrete stress at bottom fiber of the girder at hold-down point due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si} e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1405.53}{788.4} + \frac{1405.53(19.47)}{10,521.33} - \frac{1222.22(12 \text{ in./ft.})}{10,521.33}$$

$$= 1.783 + 2.601 - 1.394 = 2.99 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer stage is  $0.6 f'_{ci}$ . [STD Art.9.15.2.1]

$$f'_{ci \text{ -reqd.}} = \frac{2990}{0.6} = 4983.33 \text{ psi}$$

**A.1.7.2.9**  
**Initial Stresses at**  
**Girder End**

The initial tensile stress at the top fiber and compressive stress at the bottom fiber of the girder at the girder end section are minimized by harping the web strands at the girder end. Following the TxDOT methodology (TxDOT 2001), the web strands are incrementally raised as a unit by two inches in each trial. The iterations are repeated until the top and bottom fiber stresses satisfy the allowable stress limits, or the centroid of the topmost row of harped strands is

at a distance of 2 inches from the top fiber of the girder, in which case, the concrete strength at release is updated based on the governing stress.

The position of the harped web strands, eccentricity of strands at the girder end, top and bottom fiber stresses at the girder end, and the corresponding required concrete strengths are summarized in Table A.1.7.1. The required concrete strengths are based on allowable stress limits at transfer stage specified in STD Art.9.15.2.1 presented as follows.

$$\text{Allowable compressive stress limit} = 0.6 f'_{ci}$$

$$\text{For members with bonded reinforcement allowable tension at transfer} = 7.5 \sqrt{f'_{ci}}$$

*Table A.1.7.1. Summary of Top and Bottom Stresses at Girder End for Different Harped Strand Positions and Corresponding Required Concrete Strengths.*

| Distance of the Centroid of Topmost Row of Harped Web Strands from |                 | Eccentricity of Prestressing Strands at Girder End (in.) | Top Fiber Stress (psi) | Required Concrete strength (psi) | Bottom Fiber Stress (psi) | Required Concrete strength (psi) |
|--|-----------------|--|------------------------|----------------------------------|---------------------------|----------------------------------|
| Bottom Fiber (in.)   | Top Fiber (in.) |  |                        |                                  |                           |                                  |
| 10 (no harping)  | 44              | 19.47  | -1291.11               | 29,634.91                        | 4383.73                   | 7306.22                          |
| 12   | 42              | 19.07  | -1227.96               | 26,806.80                        | 4330.30                   | 7217.16                          |
| 14   | 40              | 18.67  | -1164.81               | 24,120.48                        | 4276.86                   | 7128.10                          |
| 16   | 38              | 18.27  | -1101.66               | 21,575.96                        | 4223.43                   | 7039.04                          |
| 18   | 36              | 17.87  | -1038.51               | 19,173.23                        | 4169.99                   | 6949.99                          |
| 20   | 34              | 17.47  | -975.35                | 16,912.30                        | 4116.56                   | 6860.93                          |
| 22   | 32              | 17.07  | -912.20                | 14,793.17                        | 4063.12                   | 6771.87                          |
| 24   | 30              | 16.67  | -849.05                | 12,815.84                        | 4009.68                   | 6682.81                          |
| 26   | 28              | 16.27  | -785.90                | 10,980.30                        | 3956.25                   | 6593.75                          |
| 28   | 26              | 15.87  | -722.75                | 9286.56                          | 3902.81                   | 6504.69                          |
| 30   | 24              | 15.47  | -659.60                | 7734.62                          | 3849.38                   | 6415.63                          |
| 32   | 22              | 15.07  | -596.45                | 6324.47                          | 3795.94                   | 6326.57                          |
| 34   | 20              | 14.67  | -533.30                | 5056.12                          | 3742.51                   | 6237.51                          |
| 36   | 18              | 14.27  | -470.15                | 3929.57                          | 3689.07                   | 6148.45                          |
| 38   | 16              | 13.87  | -407.00                | 2944.82                          | 3635.64                   | 6059.39                          |
| 40   | 14              | 13.47  | -343.85                | 2101.86                          | 3582.20                   | 5970.34                          |
| 42   | 12              | 13.07  | -280.69                | 1400.70                          | 3528.77                   | 5881.28                          |
| 44   | 10              | 12.67  | -217.54                | 841.34                           | 3475.33                   | 5792.22                          |
| 46   | 8               | 12.27  | -154.39                | 423.77                           | 3421.89                   | 5703.16                          |
| 48   | 6               | 11.87  | -91.24                 | 148.00                           | 3368.46                   | 5614.10                          |
| 50   | 4               | 11.47  | -28.09                 | 14.03                            | 3315.02                   | 5525.04                          |
| 52   | 2               | 11.07  | 35.06                  | 58.43                            | 3261.59                   | 5435.98                          |

From Table A.1.7.1, it is evident that the web strands need to be harped to the topmost position possible to control the bottom fiber stress at the girder end.

Detailed calculations for the case when 10 web strands (5 rows) are harped to the topmost location (centroid of the topmost row of harped strands is at a distance of 2 inches from the top fiber of the girder) is presented as follows.

Eccentricity of prestressing strands at the girder end (see Figure A.1.7.2)

$$e_e = 24.75 - \frac{10(2+4+6) + 8(8) + 2(10) + 2(52+50+48+46+44)}{50}$$

$$= 11.07 \text{ in.}$$

Concrete stress at the top fiber of the girder at the girder end at transfer stage:

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si} e_e}{S_t}$$

$$= \frac{1405.53}{788.4} - \frac{1405.53 (11.07)}{8902.67} = 1.783 - 1.748 = 0.035 \text{ ksi}$$

Concrete stress at the bottom fiber of the girder at the girder end at transfer stage:

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si} e_e}{S_b}$$

$$f_{bi} = \frac{1405.53}{788.4} + \frac{1405.53 (11.07)}{10,521.33} = 1.783 + 1.479 = 3.262 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer stage is  $0.6 f'_{ci}$ . [STD

Art.9.15.2.1]

$$f'_{ci \text{ -reqd.}} = \frac{3262}{0.60} = 5436.67 \text{ psi} \quad (\text{controls})$$

The required concrete strengths are updated based on the above results as follows.

Concrete strength at release,  $f'_{ci} = 5436.67 \text{ psi}$

Concrete strength at service,  $f'_c = 5590 \text{ psi}$

### **A.1.7.3 Iteration 3**

A third iteration is carried out to refine the prestress losses based on the updated concrete strengths. Based on the new prestress losses, the concrete strength at release and service will be further refined.

#### **A.1.7.3.1 Concrete Shrinkage**

[STD Art. 9.16.2.1.1]

For pretensioned members, the loss in prestress due to concrete shrinkage is given as:

$$SH = 17,000 - 150 RH \quad [\text{STD Eq. 9-4}]$$

where:

$RH$  is the relative humidity = 60 percent

$$SH = [17,000 - 150(60)] \frac{1}{1000} = 8.0 \text{ ksi}$$

#### **A.1.7.3.2 Elastic Shortening**

[STD Art. 9.16.2.1.2]

For pretensioned members, the loss in prestress due to elastic shortening is given as:

$$ES = \frac{E_s}{E_{ci}} f_{cir} \quad [\text{STD Eq. 9-6}]$$

where:

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si} e_c^2}{I} - \frac{(M_g) e_c}{I}$$

$P_{si}$  = Pretension force after allowing for the initial losses, kips

As the initial losses are dependent on the elastic shortening and steel relaxation loss, which are yet to be determined, the initial loss value of 9.27 percent obtained in the last trial (iteration 2) is taken as first estimate for the initial loss in prestress.

$$\begin{aligned} P_{si} &= (\text{number of strands})(\text{area of strand})[0.9073(0.75 f'_s)] \\ &= 50(0.153)(0.9073)(0.75)(270) = 1405.52 \text{ kips} \end{aligned}$$

$$\begin{aligned} M_g &= \text{Moment due to girder self-weight at midspan, k-ft.} \\ &= 1209.98 \text{ k-ft.} \end{aligned}$$

$$\begin{aligned} e_c &= \text{Eccentricity of the prestressing strands at the midspan} \\ &= 19.47 \text{ in.} \end{aligned}$$

$$\begin{aligned} f_{cir} &= \frac{1405.52}{788.4} + \frac{1405.52(19.47)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.47)}{260,403} \\ &= 1.783 + 2.046 - 1.086 = 2.743 \text{ ksi} \end{aligned}$$

Concrete strength at release,  $f'_{ci} = 5436.67$  psi

Modulus of elasticity of girder concrete at release is given as:

$$E_{ci} = 33(w_c)^{3/2} \sqrt{f'_{ci}} \quad [\text{STD Eq. 9-8}]$$

$$= [33(150)^{3/2} \sqrt{5436.67}] \left( \frac{1}{1000} \right) = 4470.10 \text{ ksi}$$

Modulus of elasticity of prestressing steel,  $E_s = 28,000$  ksi

Prestress loss due to elastic shortening is:

$$ES = \left[ \frac{28,000}{4470.10} \right] (2.743) = 17.18 \text{ ksi}$$

### A.1.7.3.3 Creep of Concrete

[STD Art. 9.16.2.1.3]

The loss in prestress due to creep of concrete is specified to be calculated using the following formula:

$$CR_C = 12f_{cir} - 7f_{cds} \quad [\text{STD Eq. 9-9}]$$

where:

$$f_{cds} = \frac{M_S e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

$M_{SDL}$  = Moment due to superimposed dead load at midspan section = 349.29 k-ft.

$M_S$  = Moment due to slab weight at midspan section  
= 1179.03 k-ft.

$y_{bc}$  = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder = 41.157 in.

$y_{bs}$  = Distance from center of gravity of the prestressing strands at midspan to the bottom fiber of the girder  
= 24.75 – 19.47 = 5.28 in.

$I$  = Moment of inertia of the non-composite section  
= 260,403 in.<sup>4</sup>

$I_c$  = Moment of inertia of composite section = 694,599.5 in.<sup>4</sup>

$$f_{cds} = \frac{1179.03(12 \text{ in./ft.})(19.47)}{260,403} + \frac{(349.29)(12 \text{ in./ft.})(41.157 - 5.28)}{694,599.5}$$

$$= 1.058 + 0.216 = 1.274 \text{ ksi}$$

Prestress loss due to creep of concrete is:

$$CR_C = 12(2.743) - 7(1.274) = 24.0 \text{ ksi}$$

#### **A.1.7.3.4 Relaxation of Pretensioning Steel**

[STD Art. 9.16.2.1.4]

For pretensioned members with 270 ksi low-relaxation strands, the prestress loss due to relaxation of the prestressing steel is calculated using the following formula:

$$CR_S = 5000 - 0.10 ES - 0.05(SH + CR_C) \quad [\text{STD Eq. 9-10A}]$$

$$CR_S = [5000 - 0.10(17,180) - 0.05(8000 + 24,000)] \left( \frac{1}{1000} \right)$$

$$= 1.682 \text{ ksi}$$

$$\text{Initial prestress loss} = \frac{(ES + \frac{1}{2} CR_S) 100}{0.75 f'_s}$$

$$= \frac{[17.18 + 0.5(1.682)] 100}{0.75(270)} = 8.90\% < 9.27\% \text{ (assumed value of initial prestress loss)}$$

Therefore, another trial is required assuming 8.90 percent initial prestress loss.

The change in initial prestress loss will not affect the prestress loss due to concrete shrinkage. Therefore, the next trial will involve updating the losses due to elastic shortening, steel relaxation, and creep of concrete.

Based on an initial prestress loss value of 8.90 percent, the pretension force after allowing for the initial losses is calculated as follows.

$$P_{si} = (\text{number of strands})(\text{area of each strand})[0.911(0.75 f'_s)]$$

$$= 50(0.153)(0.911)(0.75)(270) = 1411.25 \text{ kips}$$

Loss in prestress due to elastic shortening

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si} e_c^2}{I} - \frac{(M_g) e_c}{I}$$

$$= \frac{1411.25}{788.4} + \frac{1411.25(19.47)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.47)}{260,403}$$

$$= 1.790 + 2.054 - 1.086 = 2.758 \text{ ksi}$$

$$E_s = 28,000 \text{ ksi}$$

$$E_{ci} = 4470.10 \text{ ksi}$$

$$ES = \left[ \frac{28,000}{4470.10} \right] (2.758) = 17.28 \text{ ksi}$$

Loss in prestress due to creep of concrete

$$CR_C = 12f_{cir} - 7f_{cds}$$

The value of  $f_{cds}$  is independent of the initial prestressing force value and will be same as calculated in Section A.1.7.3.3.

$$f_{cds} = 1.274 \text{ ksi}$$

$$CR_C = 12(2.758) - 7(1.274) = 24.18 \text{ ksi}$$

Loss in prestress due to relaxation of steel

$$CR_S = 5000 - 0.10 ES - 0.05(SH + CR_C)$$

$$= [5000 - 0.10(17,280) - 0.05(8000 + 24,180)] \left( \frac{1}{1000} \right)$$

$$= 1.663 \text{ ksi}$$

$$\text{Initial prestress loss} = \frac{(ES + \frac{1}{2} CR_S) 100}{0.75 f'_s}$$

$$= \frac{[17.28 + 0.5(1.663)] 100}{0.75(270)} = 8.94\% \approx 8.90\% \text{ (assumed value)}$$

for initial prestress loss)

#### **A.1.7.3.5 Total Losses at Transfer**

$$\text{Total prestress loss at transfer} = (ES + \frac{1}{2} CR_S)$$

$$= [17.28 + 0.5(1.663)] = 18.11 \text{ ksi}$$

$$\text{Effective initial prestress, } f_{si} = 202.5 - 18.11 = 184.39 \text{ ksi}$$

$$P_{si} = \text{Effective pretension after allowing for the initial prestress loss}$$

$$= (\text{number of strands})(\text{area of strand})(f_{si})$$

$$= 50(0.153)(184.39) = 1410.58 \text{ kips}$$

#### **A.1.7.3.6 Total Losses at Service Loads**

$$\text{Loss in prestress due to concrete shrinkage, } SH = 8.0 \text{ ksi}$$

$$\text{Loss in prestress due to elastic shortening, } ES = 17.28 \text{ ksi}$$

$$\text{Loss in prestress due to creep of concrete, } CR_C = 24.18 \text{ ksi}$$

$$\text{Loss in prestress due to steel relaxation, } CR_S = 1.663 \text{ ksi}$$



$$\begin{aligned}\text{Total final loss in prestress} &= SH + ES + CR_C + CR_S \\ &= 8.0 + 17.28 + 24.18 + 1.663 = 51.12 \text{ ksi}\end{aligned}$$

$$\text{or } \frac{51.12(100)}{0.75(270)} = 25.24 \%$$

$$\text{Effective final prestress, } f_{se} = 0.75(270) - 51.12 = 151.38 \text{ ksi}$$

$$\begin{aligned}P_{se} &= \text{Effective pretension after allowing for the final prestress loss} \\ &= (\text{number of strands})(\text{area of strand})(\text{effective final prestress}) \\ &= 50(0.153)(151.38) = 1158.06 \text{ kips}\end{aligned}$$

#### **A.1.7.3.7** **Final Stresses at** **Midspan**

Concrete stress at top fiber of the girder at midspan section due to applied loads and effective prestress

$$\begin{aligned}f_{tf} &= \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + f_i = \frac{1158.06}{788.4} - \frac{1158.06 (19.47)}{8902.67} + 3.626 \\ &= 1.469 - 2.533 + 3.626 = 2.562 \text{ ksi}\end{aligned}$$

( $f_i$  calculations are presented in Section A.1.6.1.)

Compressive stress limit under service load combination is  $0.6 f'_c$ .  
[STD Art. 9.15.2.2]

$$f'_c \text{ -reqd.} = \frac{2562}{0.6} = 4270 \text{ psi}$$

Concrete stress at top fiber of the girder at midspan due to effective prestress + permanent dead loads

$$\begin{aligned}f_{tf} &= \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}} \\ &= \frac{1158.06}{788.4} - \frac{1158.06 (19.47)}{8902.67} + \frac{(1209.98 + 1179.03)(12 \text{ in./ft.})}{8902.67} \\ &\quad + \frac{349.29(12 \text{ in./ft.})}{54,083.9} \\ &= 1.469 - 2.533 + 3.22 + 0.077 = 2.233 \text{ ksi}\end{aligned}$$

Compressive stress limit for effective prestress + permanent dead loads =  $0.4 f'_c$   
[STD Art. 9.15.2.2]

$$f'_c \text{ -reqd.} = \frac{2233}{0.40} = 5582.5 \text{ psi} \quad (\text{controls})$$

Concrete stress at top fiber of the girder at midspan due to live load + 0.5(effective prestress + dead loads)

$$\begin{aligned}
 f_{tf} &= \frac{M_{LL+I}}{S_{tg}} + 0.5 \left( \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}} \right) \\
 &= \frac{1478.39(12 \text{ in./ft.})}{54,083.9} + 0.5 \left\{ \frac{1158.06}{788.4} - \frac{1158.06(19.47)}{8902.67} + \right. \\
 &\quad \left. \frac{(1209.98 + 1179.03)(12 \text{ in./ft.})}{8902.67} + \frac{349.29(12 \text{ in./ft.})}{54,083.9} \right\} \\
 &= 0.328 + 0.5(1.469 - 2.533 + 3.22 + 0.077) = 1.445 \text{ ksi}
 \end{aligned}$$

Allowable limit for compressive stress due to live load + 0.5(effective prestress + dead loads) =  $0.4 f'_c$  [STD Art. 9.15.2.2]

$$f'_c \text{ -reqd.} = \frac{1445}{0.40} = 3612.5 \text{ psi}$$

Tensile stress at the bottom fiber of the girder at midspan due to service loads

$$\begin{aligned}
 f_{bf} &= \frac{P_{se}}{A} + \frac{P_{se} e_c}{S_b} - f_b \text{ (} f_b \text{ calculations are presented in Sec. A.1.6.1.)} \\
 &= \frac{1158.06}{788.4} + \frac{1158.06(19.47)}{10,521.33} - 4.024 \\
 &= 1.469 + 2.143 - 4.024 = -0.412 \text{ ksi (negative sign indicates} \\
 &\quad \text{tensile stress)}
 \end{aligned}$$

For members with bonded reinforcement, allowable tension in the precompressed tensile zone =  $6\sqrt{f'_c}$ . [STD Art. 9.15.2.2]

$$f'_c \text{ -reqd.} = \left( \frac{412}{6} \right)^2 = 4715.1 \text{ psi}$$

The concrete strength at service is updated based on the final stresses at the midspan section under different loading combinations. The required concrete strength at service is determined to be 5582.5 psi.

**A.1.7.3.8**  
**Initial Stresses at**  
**Hold-Down Point**

Prestressing force after allowing for initial prestress loss

$$P_{si} = (\text{number of strands})(\text{area of strand})(\text{effective initial prestress})$$

$$= 50(0.153)(184.39) = 1410.58 \text{ kips (Effective initial prestress calculations are presented in Section A.1.7.3.5.)}$$

Initial concrete stress at the top fiber of the girder at hold-down point due to self-weight of girder and effective initial prestress

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si} e_c}{S_t} + \frac{M_g}{S_t}$$

where:

$$M_g = \text{Moment due to girder self-weight at hold-down point based on overall girder length of 109'-8"} \\ = 1222.22 \text{ k-ft. (see Section A.1.7.1.8.)}$$

$$f_{ti} = \frac{1410.58}{788.4} - \frac{1410.58(19.47)}{8902.67} + \frac{1222.22(12 \text{ in./ft.})}{8902.67}$$

$$= 1.789 - 3.085 + 1.647 = 0.351 \text{ ksi}$$

Initial concrete stress at bottom fiber of the girder at hold-down point due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si} e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1410.58}{788.4} + \frac{1410.58(19.47)}{10,521.33} - \frac{1222.22(12 \text{ in./ft.})}{10,521.33}$$

$$= 1.789 + 2.610 - 1.394 = 3.005 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer stage is  $0.6 f'_{ci}$ . [STD Art.9.15.2.1]

$$f'_{ci \text{ -reqd.}} = \frac{3005}{0.6} = 5008.3 \text{ psi}$$

**A.1.7.3.9**  
**Initial Stresses at**  
**Girder End**

The eccentricity of the prestressing strands at the girder end when 10 web strands are harped to the topmost location (centroid of the topmost row of harped strands is at a distance of 2 inches from the top fiber of the girder) is calculated as follows (see Fig. A.1.7.2.):

$$e_e = 24.75 - \frac{10(2+4+6) + 8(8) + 2(10) + 2(52+50+48+46+44)}{50}$$

$$= 11.07 \text{ in.}$$

Concrete stress at the top fiber of the girder at the girder end at transfer stage:

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si} e_e}{S_t}$$

$$= \frac{1410.58}{788.4} - \frac{1410.58 (11.07)}{8902.67} = 1.789 - 1.754 = 0.035 \text{ ksi}$$

Concrete stress at the bottom fiber of the girder at the girder end at transfer stage:

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si} e_e}{S_b}$$

$$f_{bi} = \frac{1410.58}{788.4} + \frac{1410.58 (11.07)}{10,521.33} = 1.789 + 1.484 = 3.273 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer stage is  $0.6 f'_{ci}$ . [STD

Art.9.15.2.1]

$$f'_{ci \text{ -reqd.}} = \frac{3273}{0.60} = 5455 \text{ psi (controls)}$$

The required concrete strengths are updated based on the above results as follows.

Concrete strength at release,  $f'_{ci} = 5455 \text{ psi}$

Concrete strength at service,  $f'_c = 5582.5 \text{ psi}$

The difference in the required concrete strengths at release and at service obtained from iterations 2 and 3 is less than 20 psi. Hence, the concrete strengths are sufficiently converged, and another iteration is not required.

Therefore provide  $f'_{ci} = 5455 \text{ psi}$

$$f'_c = 5582.5 \text{ psi}$$

50 – 0.5 in. diameter, 10 draped at the end, GR 270 low-relaxation strands.

The final strand patterns at the midspan section and at the girder ends are shown in Figures A.1.7.1 and A.1.7.2. The longitudinal strand profile is shown in Figure A.1.7.3.

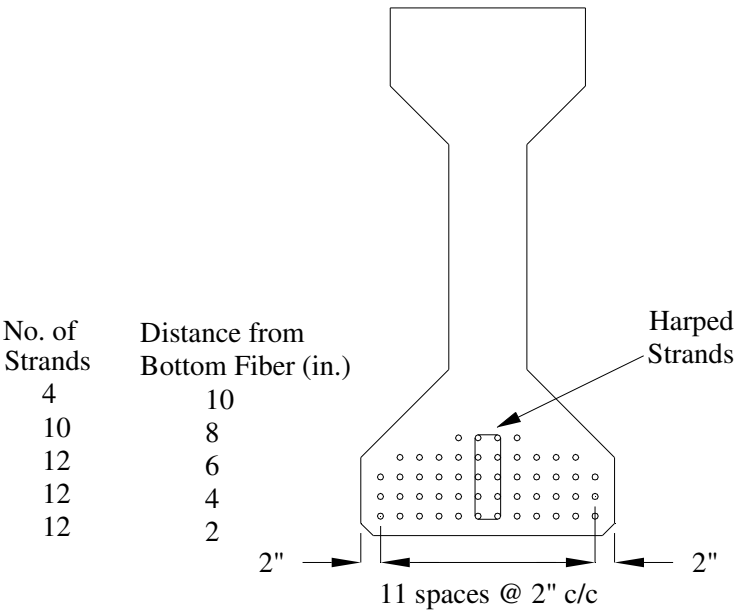


Figure A.1.7.1. Final Strand Pattern at Midspan.

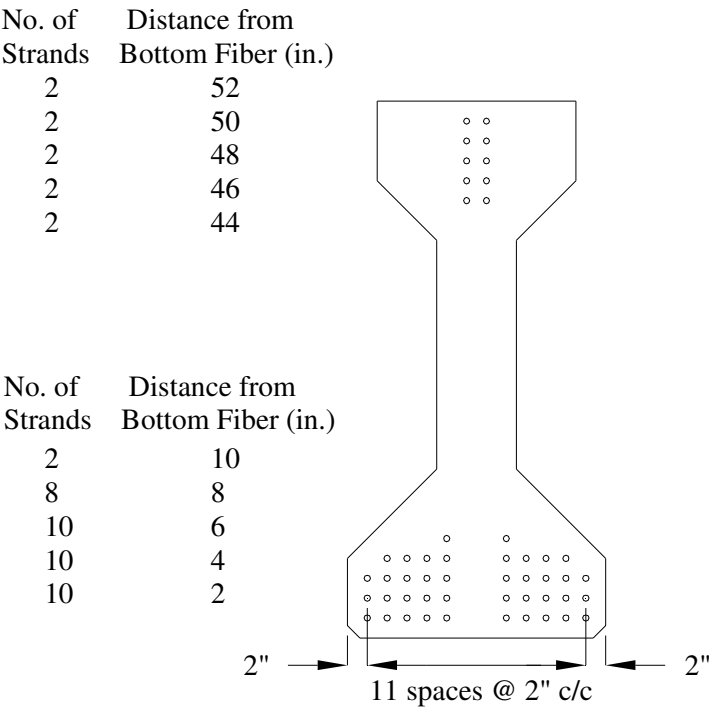


Figure A.1.7.2. Final Strand Pattern at Girder End.

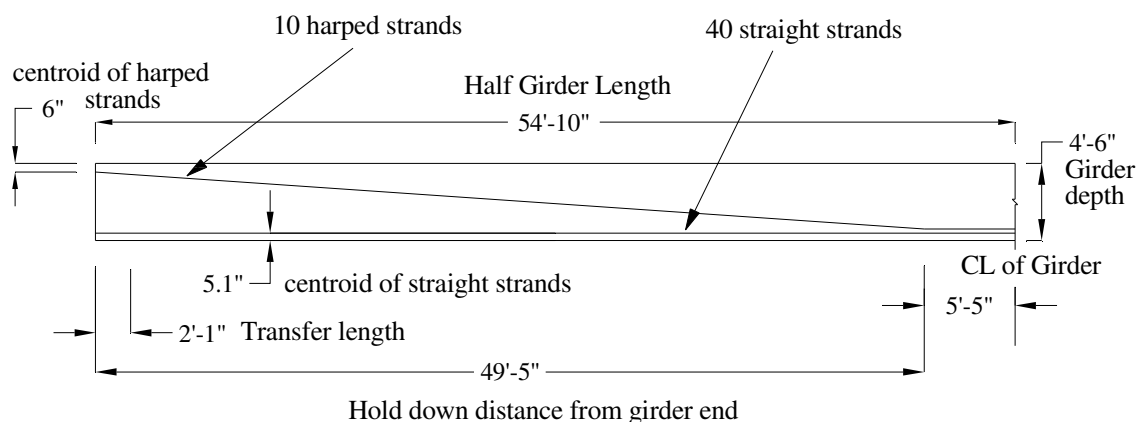


Figure A.1.7.3. Longitudinal Strand Profile (half of the girder length is shown).

The distance between the centroid of the 10 harped strands and the top fiber of the girder at the girder end

$$= \frac{2(2) + 2(4) + 2(6) + 2(8) + 2(10)}{10} = 6 \text{ in.}$$

The distance between the centroid of the 10 harped strands and the bottom fiber of the girder at the harp points

$$= \frac{2(2) + 2(4) + 2(6) + 2(8) + 2(10)}{10} = 6 \text{ in.}$$

Transfer length distance from girder end = 50 (strand diameter)

[STD Art. 9.20.2.4]

Transfer length =  $50(0.50) = 25 \text{ in.} = 2.083 \text{ ft.}$

The distance between the centroid of the 10 harped strands and the top of the girder at the transfer length section

$$= 6 \text{ in.} + \frac{(54 \text{ in} - 6 \text{ in} - 6 \text{ in})}{49.4 \text{ ft.}} (2.083 \text{ ft.}) = 7.77 \text{ in.}$$

The distance between the centroid of the 40 straight strands and the bottom fiber of the girder at all locations

$$= \frac{10(2) + 10(4) + 10(6) + 8(8) + 2(10)}{40} = 5.1 \text{ in.}$$

**A.1.8**  
**STRESS SUMMARY**

**A.1.8.1**  
**Concrete Stresses**  
**at Transfer**

**A.1.8.1.1**  
**Allowable Stress**  
**Limits**

[STD Art. 9.15.2.1]

The allowable stress limits at transfer specified by the Standard Specifications are as follows.

Compression:  $0.6 f'_{ci} = 0.6(5455) = +3273 \text{ psi} = 3.273 \text{ ksi}$   
(comp.)

Tension: The maximum allowable tensile stress is

$$7.5 \sqrt{f'_{ci}} = 7.5 \sqrt{5455} = -553.93 \text{ psi (tension)}$$

If the calculated tensile stress exceeds 200 psi or

$3 \sqrt{f'_{ci}} = 3 \sqrt{5455} = 221.57 \text{ psi}$ , whichever is smaller, bonded reinforcement should be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section.

**A.1.8.1.2**  
**Stresses at Girder**  
**End**

Stresses at the girder end are checked only at transfer, because it almost always governs.

Eccentricity of prestressing strands at the girder end when 10 web strands are harped to the topmost location (centroid of the topmost row of harped strands is at a distance of 2 inches from the top fiber of the girder)

$$e_e = 24.75 - \frac{10(2+4+6) + 8(8) + 2(10) + 2(52+50+48+46+44)}{50}$$

$$= 11.07 \text{ in.}$$

Prestressing force after allowing for initial prestress loss

$$P_{si} = (\text{number of strands})(\text{area of strand})(\text{effective initial prestress})$$

$$= 50(0.153)(184.39) = 1410.58 \text{ kips (Effective initial prestress calculations are presented in Section A.1.7.3.5.)}$$

Concrete stress at the top fiber of the girder at the girder end at transfer:

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si} e_e}{S_t}$$

$$= \frac{1410.58}{788.4} - \frac{1410.58 (11.07)}{8902.67} = 1.789 - 1.754 = +0.035 \text{ ksi}$$

Allowable Compression:  $+3.273 \text{ ksi} \gg +0.035 \text{ ksi (reqd.)}$  (O.K.)

Because the top fiber stress is compressive, there is no need for additional bonded reinforcement.

Concrete stress at the bottom fiber of the girder at the girder end at transfer stage:

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si} e_e}{S_b}$$

$$= \frac{1410.58}{788.4} + \frac{1410.58 (11.07)}{10,521.33} = 1.789 + 1.484 = +3.273 \text{ ksi}$$

Allowable compression: +3.273 ksi = +3.273 ksi (reqd.) (O.K.)

### **A.1.8.1.3** **Stresses at Transfer** **Length Section**

Stresses at transfer length are checked only at release, because it almost always governs.

$$\begin{aligned} \text{Transfer length} &= 50(\text{strand diameter}) \quad [\text{STD Art. 9.20.2.4}] \\ &= 50(0.50) = 25 \text{ in.} = 2.083 \text{ ft.} \end{aligned}$$

The transfer length section is located at a distance of 2'-1" from the end of the girder or at a point 1'-6.5" from the centerline of the bearing as the girder extends 6.5" beyond the bearing centerline. Overall girder length of 109'-8" is considered for the calculation of bending moment at transfer length.

$$\text{Moment due to girder self-weight, } M_g = 0.5wx(L - x)$$

where:

$$w = \text{Self-weight of the girder} = 0.821 \text{ kips/ft.}$$

$$L = \text{Overall girder length} = 109.67 \text{ ft.}$$

$$x = \text{Transfer length distance from girder end} = 2.083 \text{ ft.}$$

$$M_g = 0.5(0.821)(2.083)(109.67 - 2.083) = 92 \text{ k-ft.}$$

Eccentricity of prestressing strands at transfer length section

$$e_t = e_c - (e_c - e_e) \frac{(49.404 - x)}{49.404}$$

where:

$$e_c = \text{Eccentricity of prestressing strands at midspan} = 19.47 \text{ in.}$$

$$\begin{aligned} e_e &= \text{Eccentricity of prestressing strands at girder end} \\ &= 11.07 \text{ in.} \end{aligned}$$

$$x = \text{Distance of transfer length section from girder end, ft.}$$



$$e_t = 19.47 - (19.47 - 11.07) \frac{(49.404 - 2.083)}{49.404} = 11.42 \text{ in.}$$

Initial concrete stress at top fiber of the girder at transfer length section due to self-weight of girder and effective initial prestress

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si} e_t}{S_t} + \frac{M_g}{S_t}$$

$$f_{ti} = \frac{1410.58}{788.4} - \frac{1410.58(11.42)}{8902.67} + \frac{92(12 \text{ in./ft.})}{8902.67}$$

$$= 1.789 - 1.809 + 0.124 = +0.104 \text{ ksi}$$

Allowable compression: +3.273 ksi >> 0.104 ksi (reqd.) (O.K.)

Because the top fiber stress is compressive, there is no need for additional bonded reinforcement.

Initial concrete stress at bottom fiber of the girder at the transfer length section due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si} e_t}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1410.58}{788.4} + \frac{1410.58(11.42)}{10,521.33} - \frac{92(12 \text{ in./ft.})}{10,521.33}$$

$$= 1.789 + 1.531 - 0.105 = 3.215 \text{ ksi}$$

Allowable compression: +3.273 ksi > 3.215 ksi (reqd.) (O.K.)

#### **A.1.8.1.4 Stresses at Hold- Down Points**

The eccentricity of the prestressing strands at the harp points is the same as at midspan.

$$e_{harp} = e_c = 19.47 \text{ in.}$$

Initial concrete stress at top fiber of the girder at the hold-down point due to self-weight of girder and effective initial prestress

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si} e_{harp}}{S_t} + \frac{M_g}{S_t}$$

where:

$$M_g = \text{Moment due to girder self-weight at hold-down point}$$

$$= \text{based on overall girder length of } 109'-8''$$

$$= 1222.22 \text{ k-ft. (see Section A.1.7.1.8.)}$$

$$f_{ti} = \frac{1410.58}{788.4} - \frac{1410.58(19.47)}{8902.67} + \frac{1222.22(12 \text{ in./ft.})}{8902.67}$$

$$= 1.789 - 3.085 + 1.647 = 0.351 \text{ ksi}$$

Allowable compression: +3.273 ksi >> 0.351 ksi (reqd.) (O.K.)

Initial concrete stress at bottom fiber of the girder at the hold-down point due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si} e_{harp}}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1410.58}{788.4} + \frac{1410.58(19.47)}{10,521.33} - \frac{1222.22(12 \text{ in./ft.})}{10,521.33}$$

$$= 1.789 + 2.610 - 1.394 = 3.005 \text{ ksi}$$

Allowable compression: +3.273 ksi > 3.005 ksi (reqd.) (O.K.)

#### **A.1.8.1.5** **Stresses at Midspan**

Bending moment due to girder self-weight at midspan section based on overall girder length of 109'-8"

$$M_g = 0.5wx(L - x)$$

where:

- $w$  = Self-weight of the girder = 0.821 kips/ft.
- $L$  = Overall girder length = 109.67 ft.
- $x$  = Half the girder length = 54.84 ft.

$$M_g = 0.5(0.821)(54.84)(109.67 - 54.84) = 1234.32 \text{ k-ft.}$$

Initial concrete stress at top fiber of the girder at midspan section due to self-weight of the girder and the effective initial prestress

$$f_{ti} = \frac{P_{si}}{A} - \frac{P_{si} e_c}{S_t} + \frac{M_g}{S_t}$$

$$f_{ti} = \frac{1410.58}{788.4} - \frac{1410.58(19.47)}{8902.67} + \frac{1234.32(12 \text{ in./ft.})}{8902.67}$$

$$= 1.789 - 3.085 + 1.664 = 0.368 \text{ ksi}$$

Allowable compression: +3.273 ksi >> 0.368 ksi (reqd.) (O.K.)

Initial concrete stress at bottom fiber of the girder at midspan section due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_{si}}{A} + \frac{P_{si} e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1410.58}{788.4} + \frac{1410.58(19.47)}{10,521.33} - \frac{1234.32(12 \text{ in./ft.})}{10,521.33}$$

$$= 1.789 + 2.610 - 1.408 = 2.991 \text{ ksi}$$

Allowable compression: +3.273 ksi > 2.991 ksi (reqd.) (O.K.)

#### **A.1.8.1.6** **Stress Summary at** **Transfer**

Allowable Stress Limits:

Compression: + 3.273 ksi

Tension: – 0.20 ksi without additional bonded reinforcement  
– 0.554 ksi with additional bonded reinforcement

| Location                | Top of girder<br>$f_t$ (ksi) | Bottom of girder<br>$f_b$ (ksi) |
|-------------------------|------------------------------|---------------------------------|
| Girder end              | +0.035                       | +3.273                          |
| Transfer length section | +0.104                       | +3.215                          |
| Hold-down points        | +0.351                       | +3.005                          |
| Midspan                 | +0.368                       | +2.991                          |

#### **A.1.8.2** **Concrete Stresses** **at Service Loads** **A.1.8.2.1** **Allowable Stress** **Limits**

[STD Art. 9.15.2.2]

The allowable stress limits at service load after losses have occurred specified by the Standard Specifications are presented as follows.

Compression:

Case (I): For all load combinations

$$0.60 f'_c = 0.60(5582.5)/1000 = +3.349 \text{ ksi (for precast girder)}$$

$$0.60 f'_c = 0.60(4000)/1000 = +2.400 \text{ ksi (for slab)}$$

Case (II): For effective prestress + permanent dead loads

$$0.40 f'_c = 0.40(5582.5)/1000 = +2.233 \text{ ksi (for precast girder)}$$

$$0.40 f'_c = 0.40(4000)/1000 = +1.600 \text{ ksi (for slab)}$$

Case (III): For live loads + 0.5(effective prestress + dead loads)

$$0.40 f'_c = 0.40(5582.5)/1,000 = +2.233 \text{ ksi (for precast girder)}$$

$$0.40 f'_c = 0.40(4000)/1,000 = +1.600 \text{ ksi (for slab)}$$

Tension: For members with bonded reinforcement

$$6\sqrt{f'_c} = 6\sqrt{5582.5} \left( \frac{1}{1000} \right) = -0.448 \text{ ksi}$$

#### **A.1.8.2.2** **Final Stresses at** **Midspan**

Effective pretension after allowing for the final prestress loss

$$\begin{aligned} P_{se} &= (\text{number of strands})(\text{area of strand})(\text{effective final prestress}) \\ &= 50(0.153)(151.38) = 1158.06 \text{ kips} \end{aligned}$$

Case (I): Service load conditions

Concrete stress at the top fiber of the girder at the midspan section due to service loads and effective prestress

$$\begin{aligned} f_{tf} &= \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL} + M_{LL+I}}{S_{tg}} \\ &= \frac{1158.06}{788.4} - \frac{1158.06 (19.47)}{8902.67} + \frac{(1209.98 + 1179.03)(12 \text{ in./ft.})}{8902.67} \\ &\quad + \frac{(349.29 + 1478.39)(12 \text{ in./ft.})}{54,083.9} \\ &= 1.469 - 2.533 + 3.220 + 0.406 = 2.562 \text{ ksi} \end{aligned}$$

Allowable compression: +3.349 ksi > +2.562 ksi (reqd.) (O.K.)

Case (II): Effective prestress + permanent dead loads

Concrete stress at top fiber of the girder at midspan due to effective prestress + permanent dead loads

$$\begin{aligned} f_{tf} &= \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + \frac{M_g + M_S}{S_t} + \frac{M_{SDL}}{S_{tg}} \\ &= \frac{1158.06}{788.4} - \frac{1158.06 (19.47)}{8902.67} + \frac{(1209.98 + 1179.03)(12 \text{ in./ft.})}{8902.67} \\ &\quad + \frac{349.29(12 \text{ in./ft.})}{54,083.9} \\ &= 1.469 - 2.533 + 3.22 + 0.077 = 2.233 \text{ ksi} \end{aligned}$$

Allowable compression: +2.233 ksi = +2.233 ksi (reqd.) (O.K.)

Case (III): Live loads + 0.5(prestress + dead loads)

Concrete stress at top fiber of the girder at midspan due to live load + 0.5(effective prestress + dead loads)

$$\begin{aligned}
 f_{tf} &= \frac{M_{LL+I}}{S_{tg}} + 0.5 \left( \frac{P_{se}}{A} - \frac{P_{se} e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}} \right) \\
 &= \frac{1478.39(12 \text{ in./ft.})}{54,083.9} + 0.5 \left\{ \frac{1158.06}{788.4} - \frac{1158.06(19.47)}{8902.67} + \right. \\
 &\quad \left. \frac{(1209.98 + 1179.03)(12 \text{ in./ft.})}{8902.67} + \frac{349.29(12 \text{ in./ft.})}{54,083.9} \right\} \\
 &= 0.328 + 0.5(1.469 - 2.533 + 3.22 + 0.077) = 1.445 \text{ ksi}
 \end{aligned}$$

Allowable compression: +2.233 ksi > +1.445 ksi (reqd.) (O.K.)

Tensile stress at the bottom fiber of the girder at midspan due to service loads

$$\begin{aligned}
 f_{bf} &= \frac{P_{se}}{A} + \frac{P_{se} e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_{SDL} + M_{LL+I}}{S_{bc}} \\
 &= \frac{1158.06}{788.4} + \frac{1158.06(19.47)}{10,521.33} - \frac{(1209.98 + 1179.03)(12 \text{ in./ft.})}{10,521.33} \\
 &\quad - \frac{(349.29 + 1478.39)(12 \text{ in./ft.})}{16,876.83} \\
 &= 1.469 + 2.143 - 2.725 - 1.299 = -0.412 \text{ ksi (negative sign} \\
 &\quad \text{indicates tensile stress)}
 \end{aligned}$$

Allowable Tension: -0.448 ksi < -0.412 ksi (reqd.) (O.K.)

Superimposed dead and live loads contribute to the stresses at the top of the slab calculated as follows.

Case (I): Superimposed dead load and live load effect

Concrete stress at top fiber of the slab at midspan due to live load + superimposed dead loads

$$f_t = \frac{M_{SDL} + M_{LL+I}}{S_{tc}} = \frac{(349.29 + 1478.39)(12 \text{ in./ft.})}{33,325.31} = +0.658 \text{ ksi}$$

Allowable compression: +2.400 ksi > +0.658 ksi (reqd.) (O.K.)

## Case (II): Superimposed dead load effect

Concrete stress at top fiber of the slab at midspan due to superimposed dead loads

$$f_t = \frac{M_{SDL}}{S_{ic}} = \frac{(349.29)(12 \text{ in./ft.})}{33,325.31} = 0.126 \text{ ksi}$$

Allowable compression: +1.600 ksi > +0.126 ksi (reqd.) (O.K.)

## Case (III): Live load + 0.5(superimposed dead loads)

Concrete stress at top fiber of the slab at midspan due to live loads + 0.5(superimposed dead loads)

$$f_t = \frac{M_{LL+I} + 0.5(M_{SDL})}{S_{ic}}$$

$$= \frac{(1478.39)(12 \text{ in./ft.}) + 0.5(349.29)(12 \text{ in./ft.})}{33,325.31} = 0.595 \text{ ksi}$$

Allowable compression: +1.600 ksi > +0.595 ksi (reqd.) (O.K.)

### A.1.8.2.3 Summary of Stresses at Service Loads

| At Midspan | Top of slab<br>$f_t$ (ksi) | Top of Girder<br>$f_t$ (ksi) | Bottom of girder<br>$f_b$ (ksi) |
|------------|----------------------------|------------------------------|---------------------------------|
| Case I     | +0.658                     | +2.562                       | - 0.412                         |
| Case II    | +0.126                     | +2.233                       | -                               |
| Case III   | +0.595                     | +1.455                       | -                               |

### A.1.8.2.4 Composite Section Properties

The composite section properties calculated in Section A.1.4.2.4 were based on the modular ratio value of 1. But as the actual concrete strength is now selected, the actual modular ratio can be determined, and the corresponding composite section properties can be evaluated.

Modular ratio between slab and girder concrete

$$n = \left( \frac{E_{cs}}{E_{cp}} \right)$$

where:

$n$  = Modular ratio between slab and girder concrete

$E_{cs}$  = Modulus of elasticity of slab concrete, ksi

$$= 33(w_c)^{3/2} \sqrt{f'_{cs}} \quad [\text{STD Eq. 9-8}]$$

$w_c$  = Unit weight of concrete = 150 pcf

$f'_{cs}$  = Compressive strength of slab concrete at service  
= 4000 psi

$$E_{cs} = [33(150)^{3/2} \sqrt{4000}] \left( \frac{1}{1000} \right) = 3834.25 \text{ ksi}$$

$E_{cp}$  = Modulus of elasticity of precast girder concrete, ksi  
=  $33(w_c)^{3/2} \sqrt{f'_c}$

$f'_c$  = Compressive strength of precast girder concrete at service  
= 5582.5 psi

$$E_{cp} = [33(150)^{3/2} \sqrt{5582.5}] \left( \frac{1}{1000} \right) = 4529.65 \text{ ksi}$$

$$n = \frac{3834.25}{4529.65} = 0.846$$

Transformed flange width,  $b_{tf} = n \times (\text{effective flange width})$

Effective flange width = 96 in. (see Section A.1.4.2.)

$$b_{tf} = 0.846(96) = 81.22 \text{ in.}$$

Transformed flange area,  $A_{tf} = n \times (\text{effective flange width})(t_s)$

$t_s$  = Slab thickness = 8 in.

$$A_{tf} = 0.846(96)(8) = 649.73 \text{ in.}^2$$

*Table A.1.8.1. Properties of Composite Section.*

|          | Transformed Area<br>$A$ (in. <sup>2</sup> ) | $y_b$<br>in. | $Ay_b$<br>in. <sup>3</sup> | $A(y_{bc} - y_b)^2$ | $I$<br>in. <sup>4</sup> | $I + A(y_{bc} - y_b)^2$<br>in. <sup>4</sup> |
|----------|---|--------------|----------------------------|---------------------|-------------------------|---|
| Girder   | 788.40                                      | 24.75        | 19,512.9                   | 177,909.63          | 260,403.0               | 438,312.6                                   |
| Slab     | 649.73                                      | 58.00        | 37,684.3                   | 215,880.37          | 3465.4                  | 219,345.8                                   |
| $\Sigma$ | 1438.13                                     |              | 57,197.2                   |                     |                         | 657,658.4                                   |

$A_c$  = Total area of composite section = 1438.13 in.<sup>2</sup>

$h_c$  = Total height of composite section = 54 in. + 8 in. = 62 in.

$$I_c = \text{Moment of inertia of composite section} = 657,658.4 \text{ in.}^4$$

$$y_{bc} = \text{Distance from the centroid of the composite section to extreme bottom fiber of the precast girder, in.} \\ = 57,197.2/1438.13 = 39.77 \text{ in.}$$

$$y_{tg} = \text{Distance from the centroid of the composite section to extreme top fiber of the precast girder, in.} \\ = 54 - 39.772 = 14.23 \text{ in.}$$

$$y_{tc} = \text{Distance from the centroid of the composite section to extreme top fiber of the slab} = 62 - 39.77 = 22.23 \text{ in.}$$

$$S_{bc} = \text{Section modulus of composite section referenced to the extreme bottom fiber of the precast girder, in.}^3 \\ = I_c/y_{bc} = 657,658.4/39.77 = 16,535.71 \text{ in.}^3$$

$$S_{tg} = \text{Section modulus of composite section referenced to the top fiber of the precast girder, in.}^3 \\ = I_c/y_{tg} = 657,658.4/14.23 = 46,222.83 \text{ in.}^3$$

$$S_{tc} = \text{Section modulus of composite section referenced to the top fiber of the slab, in.}^3 \\ = I_c/y_{tc} = 657,658.4/22.23 = 29,586.93 \text{ in.}^3$$

### **A.1.9 FLEXURAL STRENGTH**

[STD Art. 9.17]

The flexural strength limit state is investigated for Group I loading as follows.

The Group I load factor design combination specified by the Standard Specifications is:

$$M_u = 1.3[M_g + M_S + M_{SDL} + 1.67(M_{LL+I})] \quad [\text{STD Table 3.22.1.A}]$$

where:

$$M_u = \text{Design flexural moment at midspan of the girder, k-ft.}$$

$$M_g = \text{Moment due to self-weight of the girder at midspan} \\ = 1209.98 \text{ k-ft.}$$

$$M_S = \text{Moment due to slab weight at midspan} = 1179.03 \text{ k-ft.}$$

$$M_{SDL} = \text{Moment due to superimposed dead loads at midspan} \\ = 349.29 \text{ k-ft.}$$

$$M_{LL+I} = \text{Moment due to live loads including impact loads at midspan} = 1478.39 \text{ k-ft.}$$



Substituting the moment values from Table A.1.5.1 and A.1.5.2

$$\begin{aligned} M_u &= 1.3[1209.98 + 1179.03 + 349.29 + 1.67(1478.39)] \\ &= 6769.37 \text{ k-ft.} \end{aligned}$$

For bonded members, the average stress in the pretensioning steel at ultimate load conditions is given as:

$$f_{su}^* = f_s' \left( 1 - \frac{\gamma^*}{\beta_1} \rho^* \frac{f_s'}{f_c'} \right) \quad [\text{STD Eq. 9-17}]$$

The above equation is applicable when the effective prestress after losses,  $f_{se} > 0.5 f_s'$

where:

$f_{su}^*$  = Average stress in the pretensioning steel at ultimate load,  
ksi

$f_s'$  = Ultimate stress in prestressing strands = 270 ksi

$f_{se}$  = Effective final prestress (see Section A.1.7.3.6)  
= 151.38 ksi > 0.5(270) = 135 ksi (O.K.)

The equation for  $f_{su}^*$  shown above is applicable.

$f_c'$  = Compressive strength of slab concrete at service  
= 4000 psi

$\gamma^*$  = Factor for type of prestressing steel  
= 0.28 for low-relaxation steel strands [STD Art. 9.1.2]

$$\beta_1 = 0.85 - 0.05 \frac{(f_c' - 4000)}{1000} \geq 0.65 \quad [\text{STD Art. 8.16.2.7}]$$

It is assumed that the neutral axis lies in the slab, and hence the  $f_c'$  of slab concrete is used for the calculation of the factor  $\beta_1$ . If the neutral axis is found to be lying below the slab,  $\beta_1$  will be updated.

$$\beta_1 = 0.85 - 0.05 \frac{(4000 - 4000)}{1000} = 0.85$$

$$\rho^* = \text{Ratio of prestressing steel} = \frac{A_s^*}{b d}$$

$$A_s^* = \text{Area of pretensioned reinforcement, in.}^2 \\ = (\text{number of strands})(\text{area of strand}) = 50(0.153) = 7.65 \text{ in.}^2$$

$$b = \text{Effective flange (composite slab) width} = 96 \text{ in.}$$

$$y_{bs} = \text{Distance from centroid of the strands to the bottom fiber of the girder at midspan} = 5.28 \text{ in. (see Section A.1.7.3.3)}$$

$$d = \text{Distance from top of the slab to the centroid of prestressing strands, in.} \\ = \text{girder depth } (h) + \text{slab thickness } (t_s) - y_{bs} \\ = 54 + 8 - 5.28 = 56.72 \text{ in.}$$

$$\rho^* = \frac{7.65}{96(56.72)} = 0.001405$$

$$f_{su}^* = 270 \left[ 1 - \left( \frac{0.28}{0.85} \right) (0.001405) \left( \frac{270.0}{4.0} \right) \right] = 261.565 \text{ ksi}$$

Depth of equivalent rectangular compression block

$$a = \frac{A_s^* f_{su}^*}{0.85 f'_c b} = \frac{7.65 (261.565)}{0.85(4)(96)} \\ = 6.13 \text{ in.} < t_s = 8.0 \text{ in.} \quad [\text{STD Art. 9.17.2}]$$

The depth of compression block is less than the flange (slab) thickness. Hence, the section is designed as a rectangular section, and  $f'_c$  of the slab concrete is used for calculations.

For rectangular section behavior, the design flexural strength is given as:

$$\phi M_n = \phi \left[ A_s^* f_{su}^* d \left( 1 - 0.6 \frac{\rho^* f_{su}^*}{f'_c} \right) \right] \quad [\text{STD Eq. 9-13}]$$

where:

$$\phi = \text{Strength reduction factor} = 1.0 \text{ for prestressed concrete members} \quad [\text{STD Art. 9.14}]$$

$M_n$  = Nominal moment strength of the section

$$\phi M_n = 1.0 \left[ (7.65)(261.565) \frac{(56.72)}{(12 \text{ in./ft.})} \left( 1 - 0.6 \frac{0.001405(261.565)}{4.0} \right) \right] \\ = 8936.56 \text{ k-ft.} > M_u = 6769.37 \text{ k-ft.} \quad (\text{OK})$$

**A.1.10  
DUCTILITY LIMITS**

[STD Art. 9.18]

**A.1.10.1  
Maximum  
Reinforcement**

[STD Art. 9.18.1]

To ensure that steel is yielding as ultimate capacity is approached, the reinforcement index for a rectangular section shall be such that:

$$\frac{\rho^* f_{su}^*}{f_c} < 0.36\beta_1 \quad [\text{STD Eq. 9.20}]$$

$$0.001405 \left( \frac{261.565}{4.0} \right) = 0.092 < 0.36(0.85) = 0.306 \quad (\text{O.K.})$$

**A.1.10.2  
Minimum  
Reinforcement**

[STD Art. 9.18.2]

The nominal moment strength developed by the prestressed and nonprestressed reinforcement at the critical section shall be at least 1.2 times the cracking moment,  $M_{cr}^*$

$$\phi M_n \geq 1.2 M_{cr}^*$$

$$M_{cr}^* = (f_r + f_{pe}) S_{bc} - M_{d-nc} \left( \frac{S_{bc}}{S_b} - 1 \right) \quad [\text{STD Art. 9.18.2.1}]$$

where:

$$\begin{aligned} f_r &= \text{Modulus of rupture of concrete} = 7.5 \sqrt{f_c'} \text{ for normal} \\ &\quad \text{weight concrete, ksi} \quad [\text{STD Art. 9.15.2.3}] \\ &= 7.5 \sqrt{5582.5} \left( \frac{1}{1000} \right) = 0.5604 \text{ ksi} \end{aligned}$$

$f_{pe}$  = Compressive stress in concrete due to effective prestress forces only at extreme fiber of section where tensile stress is caused by externally applied loads, ksi

The tensile stresses are caused at the bottom fiber of the girder under service loads. Therefore  $f_{pe}$  is calculated for the bottom fiber of the girder as follows.

$$f_{pe} = \frac{P_{se}}{A} + \frac{P_{se} e_c}{S_b}$$

$P_{se}$  = Effective prestress force after losses = 1158.06 kips

$e_c$  = Eccentricity of prestressing strands at midspan = 19.47 in.

$$f_{pe} = \frac{1158.06}{788.4} + \frac{1158.06(19.47)}{10,521.33} = 1.469 + 2.143 = 3.612 \text{ ksi}$$

$$M_{d-nc} = \text{Non-composite dead load moment at midspan due to self-weight of girder and weight of slab} \\ = 1209.98 + 1179.03 = 2389.01 \text{ k-ft.} = 28,668.12 \text{ k-in.}$$

$$S_b = \text{Section modulus of the precast section referenced to the extreme bottom fiber of the non-composite precast girder} = 10,521.33 \text{ in.}^3$$

$$S_{bc} = \text{Section modulus of the composite section referenced to the extreme bottom fiber of the precast girder} \\ = 16,535.71 \text{ in.}^3$$

$$M_{cr}^* = (0.5604 + 3.612)(16,535.71) - (28,668.12) \left( \frac{16,535.71}{10,521.33} - 1 \right) \\ = 68,993.6 - 16,387.8 = 52,605.8 \text{ k-in.} = 4383.8 \text{ k-ft.}$$

$$1.2 M_{cr}^* = 1.2(4383.8) = 5260.56 \text{ k-ft.} < \phi M_n = 8936.56 \text{ k-ft.} \quad (\text{O.K.})$$

### **A.1.11 SHEAR DESIGN**

[STD Art. 9.20]

The shear design for the AASHTO Type IV girder based on the Standard Specifications is presented in the following section.

Prestressed concrete members subject to shear shall be designed so that:

$$V_u < \phi (V_c + V_s) \quad [\text{STD Eq. 9-26}]$$

where:

$V_u$  = Factored shear force at the section considered (calculated using load combination causing maximum shear force), kips

$V_c$  = Nominal shear strength provided by concrete, kips

$V_s$  = Nominal shear strength provided by web reinforcement, kips

$\phi$  = Strength reduction factor for shear = 0.90 for prestressed concrete members [STD Art. 9.14]

The critical section for shear is located at a distance  $h/2$  ( $h$  is the depth of composite section) from the face of the support. However, as the support dimensions are unknown, the critical section for shear is conservatively calculated from the centerline of the bearing support. [STD Art. 9.20.1.4]

Distance of critical section for shear from bearing centerline

$$= h/2 = \frac{62}{2(12 \text{ in./ft.})} = 2.583 \text{ ft.}$$

From Tables A.1.5.1 and A.1.5.2, the shear forces at the critical section are as follows:

$$\begin{aligned} V_d &= \text{Shear force due to total dead load at the critical section} \\ &= 96.07 \text{ kips} \end{aligned}$$

$$\begin{aligned} V_{LL+I} &= \text{Shear force due to live load including impact at critical} \\ &\text{section} = 56.60 \text{ kips} \end{aligned}$$

The shear design is based on Group I loading, presented as follows.

Group I load factor design combination specified by the Standard Specifications is:

$$\begin{aligned} V_u &= 1.3(V_d + 1.67 V_{LL+I}) \\ &= 1.3[96.07 + 1.67(56.6)] = 247.8 \text{ kips} \end{aligned}$$

Shear strength provided by normal weight concrete,  $V_c$ , shall be taken as the lesser of the values  $V_{ci}$  or  $V_{cw}$ . [STD Art. 9.20.2]

Computation of  $V_{ci}$  [STD Art. 9.20.2.2]

$$V_{ci} = 0.6\sqrt{f'_c} b' d + V_d + \frac{V_i M_{cr}}{M_{max}} \geq 1.7\sqrt{f'_c} b' d \quad [\text{STD Eq. 9-27}]$$

where

$$\begin{aligned} V_{ci} &= \text{Nominal shear strength provided by concrete when} \\ &\text{diagonal cracking results from combined shear and} \\ &\text{moment, kips} \end{aligned}$$

$$\begin{aligned} f'_c &= \text{Compressive strength of girder concrete at service} \\ &= 5582.5 \text{ psi} \end{aligned}$$

$$b' = \text{Width of the web of a flanged member} = 8 \text{ in.}$$

$$\begin{aligned} d &= \text{Distance from the extreme compressive fiber to centroid} \\ &\text{of pretensioned reinforcement, but not less than } 0.8h_c \\ &= h_c - (y_b - e_x) \quad [\text{STD Art. 9.20.2.2}] \end{aligned}$$

$$h_c = \text{Depth of composite section} = 62 \text{ in.}$$

$$\begin{aligned} y_b &= \text{Distance from centroid to the extreme bottom fiber of} \\ &\text{the non-composite precast girder} = 24.75 \text{ in.} \end{aligned}$$

$e_x$  = Eccentricity of prestressing strands at the critical section for shear

$$= e_c - (e_c - e_e) \frac{(49.404 - x)}{49.404}$$

$e_c$  = Eccentricity of prestressing strands at midspan  
= 19.12 in.

$e_e$  = Eccentricity of prestressing strands at the girder end  
= 11.07 in.

$x$  = Distance of critical section from girder end = 2.583 ft.

$$e_x = 19.47 - (19.47 - 11.07) \frac{(49.404 - 2.583)}{49.404} = 11.51 \text{ in.}$$

$d$  =  $62 - (24.75 - 11.51) = 48.76 \text{ in.}$   
 $= 0.8h_c = 0.8(62) = 49.6 \text{ in.} > 48.76 \text{ in.}$   
 Therefore  $d = 49.6 \text{ in.}$  is used in further calculations.

$V_d$  = Shear force due to total dead load at the critical section  
= 96.07 kips

$V_i$  = Factored shear force at the section due to externally applied loads occurring simultaneously with maximum moment,  $M_{max}$   
 $= V_{mu} - V_d$

$V_{mu}$  = Factored shear force occurring simultaneously with factored moment  $M_u$ , conservatively taken as design shear force at the section,  $V_u = 247.8 \text{ kips}$

$$V_i = 247.8 - 96.07 = 151.73 \text{ kips}$$

$M_{max}$  = Maximum factored moment at the critical section due to externally applied loads  
 $= M_u - M_d$

$M_d$  = Bending moment at the critical section due to unfactored dead load = 254.36 k-ft. (see Table A.1.5.1)

$M_{LL+I}$  = Bending moment at the critical section due to live load including impact = 146.19 k-ft. (see Table A.1.5.2)

$$\begin{aligned}
 M_u &= \text{Factored bending moment at the section} \\
 &= 1.3(M_d + 1.67M_{LL+I}) \\
 &= 1.3[254.36 + 1.67(146.19)] = 648.05 \text{ k-ft.}
 \end{aligned}$$

$$M_{max} = 648.05 - 254.36 = 393.69 \text{ k-ft.}$$

$$\begin{aligned}
 M_{cr} &= \text{Moment causing flexural cracking at the section due to} \\
 &\quad \text{externally applied loads} \\
 &= \frac{I}{Y_t} (6\sqrt{f'_c} + f_{pe} - f_d) \quad [\text{STD Eq. 9-28}]
 \end{aligned}$$

$$\begin{aligned}
 f_{pe} &= \text{Compressive stress in concrete due to effective prestress} \\
 &\quad \text{at the extreme fiber of the section where tensile stress is} \\
 &\quad \text{caused by externally applied loads, which is the bottom} \\
 &\quad \text{fiber of the girder in the present case}
 \end{aligned}$$

$$= \frac{P_{se}}{A} + \frac{P_{se}e_x}{S_b}$$

$$P_{se} = \text{Effective final prestress} = 1158.06 \text{ kips}$$

$$f_{pe} = \frac{1158.06}{788.4} + \frac{1158.06(11.51)}{10,521.33} = 1.469 + 1.267 = 2.736 \text{ ksi}$$

$$\begin{aligned}
 f_d &= \text{Stress due to unfactored dead load at extreme fiber of} \\
 &\quad \text{the section where tensile stress is caused by externally} \\
 &\quad \text{applied loads, which is the bottom fiber of the girder in} \\
 &\quad \text{the present case}
 \end{aligned}$$

$$= \left[ \frac{M_g + M_s}{S_b} + \frac{M_{SDL}}{S_{bc}} \right]$$

$$M_g = \text{Moment due to self-weight of the girder at the critical section} = 112.39 \text{ k-ft. (see Table A.1.5.1)}$$

$$\begin{aligned}
 M_s &= \text{Moment due to slab weight at the critical section} \\
 &= 109.52 \text{ k-ft. (see Table A.1.5.1)}
 \end{aligned}$$

$$M_{SDL} = \text{Moment due to superimposed dead loads at the critical section} = 32.45 \text{ k-ft.}$$

$$S_b = \text{Section modulus referenced to the extreme bottom fiber of the non-composite precast girder} = 10,521.33 \text{ in.}^3$$

$$\begin{aligned}
 S_{bc} &= \text{Section modulus of the composite section referenced to} \\
 &\quad \text{the extreme bottom fiber of the precast girder} \\
 &= 16,535.71 \text{ in.}^3
 \end{aligned}$$

$$f_d = \left[ \frac{(112.39 + 109.52)(12 \text{ in./ft.})}{10,521.33} + \frac{32.45(12 \text{ in./ft.})}{16,535.71} \right]$$

$$= 0.253 + 0.024 = 0.277 \text{ ksi}$$

$I$  = Moment of inertia about the centroid of the cross-section = 657,658.4 in.<sup>4</sup>

$Y_t$  = Distance from centroidal axis of composite section to the extreme fiber in tension, which is the bottom fiber of the girder in the present case = 39.77 in.

$$M_{cr} = \frac{657,658.4}{39.772} \left( \frac{6\sqrt{5582.5}}{1000} + 2.736 - 0.277 \right)$$

$$= 48,074.23 \text{ k-in.} = 4006.19 \text{ k-ft.}$$

$$V_{ci} = \frac{0.6\sqrt{5582.5}}{1000}(8)(49.6) + 96.07 + \frac{151.73(4006.19)}{393.69}$$

$$= 17.79 + 96.07 + 1544.00 = 1657.86 \text{ kips}$$

$$\text{Minimum } V_{ci} = 1.7\sqrt{f'_c} b'd \quad [\text{STD Art. 9.20.2.2}]$$

$$= \frac{1.7\sqrt{5582.5}}{1000}(8)(49.6)$$

$$= 50.40 \text{ kips} \ll V_{ci} = 1657.86 \text{ kips} \quad (\text{O.K.})$$

Computation of  $V_{cw}$ : [STD Art. 9.20.2.3]

$$V_{cw} = (3.5\sqrt{f'_c} + 0.3 f_{pc}) b' d + V_p \quad [\text{STD Eq. 9-29}]$$

where:

$V_{cw}$  = Nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web, kips

$f_{pc}$  = Compressive stress in concrete at centroid of cross-section resisting externally applied loads, ksi

$$= \frac{P_{se}}{A} - \frac{P_{se} e_x (y_{bcomp} - y_b)}{I} + \frac{M_D (y_{bcomp} - y_b)}{I}$$

$P_{se}$  = Effective final prestress = 1158.06 kips



$e_x$  = Eccentricity of prestressing strands at the critical section for shear = 11.51 in.

$y_{bcomp}$  = Lesser of  $y_{bc}$  and  $y_w$ , in.

$y_{bc}$  = Distance from centroid of the composite section to the extreme bottom fiber of the precast girder = 39.77 in.

$y_w$  = Distance from bottom fiber of the girder to the junction of the web and top flange  
 $= h - t_f - t_{fil}$

$h$  = Depth of precast girder = 54 in.

$t_f$  = Thickness of girder flange = 8 in.

$t_{fil}$  = Thickness of girder fillets = 6 in.

$y_w$  =  $54 - 8 - 6 = 40$  in.  $> y_{bc} = 39.77$  in.

Therefore  $y_{bcomp} = 39.77$  in.

$y_b$  = Distance from centroid to the extreme bottom fiber of the non-composite precast girder = 24.75 in.

$M_D$  = Moment due to unfactored non-composite dead loads at the critical section  
 $= 112.39 + 109.52 = 221.91$  k-ft. (see Table A.1.5.1)

$$\begin{aligned} f_{pc} &= \frac{1158.06}{788.4} - \frac{1158.06(11.51)(39.772 - 24.75)}{260,403} \\ &\quad + \frac{221.91(12 \text{ in./ft.})(39.772 - 24.75)}{260,403} \\ &= 1.469 - 0.769 + 0.154 = 0.854 \text{ ksi} \end{aligned}$$

$b'$  = Width of the web of a flanged member = 8 in.

$d$  = Distance from the extreme compressive fiber to centroid of pretensioned reinforcement = 49.6 in.

$V_p$  = Vertical component of prestress force for harped strands, kips  
 $= P_{se} \sin \Psi$

$$\begin{aligned}
 P_{se} &= \text{Effective prestress force for the harped strands, kips} \\
 &= (\text{number of harped strands})(\text{area of strand})(\text{effective final prestress}) \\
 &= 10(0.153)(151.38) = 231.61 \text{ kips}
 \end{aligned}$$

$$\begin{aligned}
 \Psi &= \text{Angle of harped tendons to the horizontal, radians} \\
 &= \tan^{-1} \left( \frac{h - y_{ht} - y_{hb}}{0.5(HD_e)} \right)
 \end{aligned}$$

$$y_{ht} = \text{Distance of the centroid of the harped strands from top fiber of the girder at girder end} = 6 \text{ in. (see Fig. A.1.7.3)}$$

$$y_{hb} = \text{Distance of the centroid of the web strands from bottom fiber of the girder at hold-down point} = 6 \text{ in. (see Figure A.1.7.3)}$$

$$\begin{aligned}
 HD_e &= \text{Distance of hold-down point from the girder end} \\
 &= 49.404 \text{ ft. (see Figure A.1.7.3)}
 \end{aligned}$$

$$\Psi = \tan^{-1} \left( \frac{54 - 6 - 6}{49.404 (12 \text{ in./ft.})} \right) = 0.071 \text{ radians}$$

$$V_p = 231.61 \sin (0.071) = 16.43 \text{ kips}$$

$$V_{cw} = \left( \frac{3.5\sqrt{5582.5}}{1000} + 0.3(0.854) \right) (8)(49.6) + 16.43 = 221.86 \text{ kips}$$

The allowable nominal shear strength provided by concrete,  $V_c$  is the lesser of  $V_{ci} = 1657.86$  kips and  $V_{cw} = 221.86$  kips

Therefore,  $V_c = 221.86$  kips

Shear reinforcement is not required if  $2V_u \leq \phi V_c$ .

[STD Art. 9.20]

where:

$$\begin{aligned}
 V_u &= \text{Factored shear force at the section considered (calculated using load combination causing maximum shear force)} \\
 &= 247.8 \text{ kips}
 \end{aligned}$$

$$\phi = \text{Strength reduction factor for shear} = 0.90 \text{ for prestressed concrete members} \quad [\text{STD Art. 9.14}]$$

$$V_c = \text{Nominal shear strength provided by concrete} = 221.86 \text{ kips}$$

$$2 V_u = 2 \times (247.8) = 495.6 \text{ kips} > \phi V_c = 0.9 \times (221.86) = 199.67 \text{ kips}$$

Therefore, shear reinforcement is required. The required shear reinforcement is calculated using the following criterion.

$$V_u < \phi (V_c + V_s) \quad [\text{STD Eq. 9-26}]$$

where  $V_s$  is the nominal shear strength provided by web reinforcement, kips

$$\text{Required } V_s = \frac{V_u}{\phi} - V_c = \frac{247.8}{0.9} - 221.86 = 53.47 \text{ kips}$$

Maximum shear force that can be carried by reinforcement

$$V_{s \max} = 8 \sqrt{f'_c} b' d \quad [\text{STD Art. 9.20.3.1}]$$

where:

$$\begin{aligned} f'_c &= \text{Compressive strength of girder concrete at service} \\ &= 5582.5 \text{ psi} \end{aligned}$$

$$\begin{aligned} V_{s \max} &= \frac{8 \sqrt{5582.5}}{1000} (8)(49.6) \\ &= 237.18 \text{ kips} > \text{Required } V_s = 53.47 \text{ kips} \quad (\text{OK}) \end{aligned}$$

The section depth is adequate for shear.

The required area of shear reinforcement is calculated using the following formula: [STD Art. 9.20.3.1]

$$V_s = \frac{A_v f_y d}{s} \quad \text{or} \quad \frac{A_v}{s} = \frac{V_s}{f_y d} \quad [\text{STD Eq. 9-30}]$$

where:

$$A_v = \text{Area of web reinforcement, in.}^2$$

$$s = \text{Center-to-center spacing of the web reinforcement, in.}$$

$$f_y = \text{Yield strength of web reinforcement} = 60 \text{ ksi}$$

$$\text{Required } \frac{A_v}{s} = \frac{(53.47)}{(60)(49.6)} = 0.018 \text{ in.}^2/\text{in.}$$

Minimum shear reinforcement [STD Art. 9.20.3.3]

$$A_{v-min} = \frac{50 b' s}{f_y} \text{ or } \frac{A_{v-min}}{s} = \frac{50 b'}{f_y} \quad [\text{STD Eq. 9-31}]$$

$$\frac{A_{v-min}}{s} = \frac{(50)(8)}{60,000} = 0.0067 \text{ in.}^2/\text{in.} < \text{Required } \frac{A_v}{s} = 0.018 \text{ in.}^2/\text{in.}$$

Therefore, provide  $\frac{A_v}{s} = 0.018 \text{ in.}^2/\text{in.}$

Typically TxDOT uses double legged #4 Grade 60 stirrups for shear reinforcement. The same is used in this design.

$$A_v = \text{Area of web reinforcement, in.}^2 = (\text{number of legs})(\text{area of bar}) \\ = 2(0.20) = 0.40 \text{ in.}^2$$

Center-to-center spacing of web reinforcement

$$s = \frac{A_v}{\text{Required } \frac{A_v}{s}} = \frac{0.40}{0.018} = 22.22 \text{ in. say } 22 \text{ in.}$$

$$V_s \text{ provided} = \frac{A_v f_y d}{s} = \frac{(0.40)(60)(49.6)}{22} = 54.1 \text{ kips}$$

Maximum spacing of web reinforcement is specified to be the lesser of  $0.75 h_c$  or 24 in., unless  $V_s$  exceeds  $4\sqrt{f'_c} b' d$ .

[STD Art. 9.20.3.2]

$$4\sqrt{f'_c} b' d = \frac{4\sqrt{5582.5}}{1000} (8)(49.6) \\ = 118.59 \text{ kips} < V_s = 54.1 \text{ kips} \quad (\text{O.K.})$$

Since  $V_s$  is less than the limit, maximum spacing of web reinforcement is given as:

$$s_{max} = \text{Lesser of } 0.75 h_c \text{ or } 24 \text{ in.}$$

where:

$h_c$  = Overall depth of the section = 62 in. (Note that the wearing surface thickness can also be included in the overall section depth calculations for shear. In the present case, the wearing surface thickness of 1.5 in. includes the future wearing surface thickness, and the actual wearing surface thickness is not specified. Therefore, the wearing surface thickness is not included. This will not have any effect on the design.)

$$s_{max} = 0.75(62) = 46.5 \text{ in.} > 24 \text{ in.}$$

Therefore maximum spacing of web reinforcement is  $s_{max} = 24 \text{ in.}$

Spacing provided,  $s = 22 \text{ in.} < s_{max} = 24 \text{ in.}$  (O.K.)

Therefore, use # 4, double-legged stirrups at 22 in. center-to-center spacing at the critical section.

The calculations presented above provide the shear design at the critical section. Different suitable sections along the span can be designed for shear using the same approach.

### **A.1.12 HORIZONTAL SHEAR DESIGN**

[STD Art. 9.20.4]

The composite flexural members are required to be designed to fully transfer the horizontal shear forces at the contact surfaces of interconnected elements.

The critical section for horizontal shear is at a distance of  $h_c/2$  (where  $h_c$  is the depth of composite section = 62 in.) from the face of the support. However, as the dimensions of the support are unknown in the present case, the critical section for shear is conservatively calculated from the centerline of the bearing support.

Distance of critical section for horizontal shear from bearing centerline:

$$h_c/2 = \frac{62 \text{ in.}}{2(12 \text{ in./ft.})} = 2.583 \text{ ft.}$$

The cross-sections subject to horizontal shear shall be designed such that:

$$V_u \leq \phi V_{nh} \quad [\text{STD Eq. 9-31a}]$$

where:

$V_u$  = Factored shear force at the section considered (calculated using load combination causing maximum shear force)  
= 247.8 kips

$V_{nh}$  = Nominal horizontal shear strength of the section, kips

$\phi$  = Strength reduction factor for shear = 0.90 for prestressed concrete members [STD Art. 9.14]

$$\text{Required } V_{nh} \geq \frac{V_u}{\phi} = \frac{247.8}{0.9} = 275.33 \text{ kips}$$

The nominal horizontal shear strength of the section,  $V_{nh}$ , is determined based on one of the following applicable cases.

Case (a): When the contact surface is clean, free of laitance, and intentionally roughened, the allowable shear force in pounds is given as:

$$V_{nh} = 80 b_v d \quad [\text{STD Art. 9.20.4.3}]$$

where:

$b_v$  = Width of cross-section at the contact surface being investigated for horizontal shear = 20 in. (top flange width of the precast girder)

$d$  = Distance from the extreme compressive fiber to centroid of pretensioned reinforcement  
 $= h_c - (y_b - e_x)$  [STD Art. 9.20.2.2]

$h_c$  = Depth of the composite section = 62 in.

$y_b$  = Distance from centroid to the extreme bottom fiber of the non-composite precast girder = 24.75 in.

$e_x$  = Eccentricity of prestressing strands at the critical section  
 $= 11.51$  in.

$d = 62 - (24.75 - 11.51) = 48.76$  in.

$$\begin{aligned} V_{nh} &= \frac{80(20)(48.76)}{1000} \\ &= 78.02 \text{ kips} < \text{Required } V_{nh} = 275.33 \text{ kips} \quad (\text{N.G.}) \end{aligned}$$

Case (b): When minimum ties are provided and contact surface is clean, free of laitance but not intentionally roughened, the allowable shear force in pounds is given as:

$$V_{nh} = 80 b_v d \quad [\text{STD Art. 9.20.4.3}]$$

$$\begin{aligned} V_{nh} &= \frac{80(20)(48.76)}{1000} \\ &= 78.02 \text{ kips} < \text{Required } V_{nh} = 275.33 \text{ kips} \quad (\text{N.G.}) \end{aligned}$$

Case (c): When minimum ties are provided and contact surface is clean, free of laitance and intentionally roughened to a full amplitude of approximately 0.25 in., the allowable shear force in pounds is given as:

$$V_{nh} = 350 b_v d \quad [\text{STD Art. 9.20.4.3}]$$

$$\begin{aligned} V_{nh} &= \frac{350(20)(48.76)}{1000} \\ &= 341.32 \text{ kips} > \text{Required } V_{nh} = 275.33 \text{ kips} \quad (\text{O.K.}) \end{aligned}$$

Design of ties for horizontal shear [STD Art. 9.20.4.5]

Minimum area of ties between the interconnected elements

$$A_{vh} = \frac{50 b_v s}{f_y}$$

where:

$A_{vh}$  = Area of horizontal shear reinforcement, in.<sup>2</sup>

$s$  = Center-to-center spacing of the web reinforcement taken as 22 in. This is the center-to-center spacing of web reinforcement, which can be extended into the slab.

$f_y$  = Yield strength of web reinforcement = 60 ksi

$$A_{vh} = \frac{50(20)(22)}{60,000} = 0.37 \text{ in.}^2 \approx 0.40 \text{ in.}^2 \text{ (area of web reinforcement provided)}$$

Maximum spacing of ties shall be:

$s$  = Lesser of 4(least web width) and 24 in. [STD Art. 9.20.4.5.a]

Least web width = 8 in.

$s = 4(8 \text{ in.}) = 32 \text{ in.} > 24 \text{ in.}$  Therefore, use maximum  $s = 24 \text{ in.}$

Maximum spacing of ties = 24 in., which is greater than the provided spacing of ties = 22 in. (O.K.)

Therefore, the provided web reinforcement shall be extended into the CIP slab to satisfy the horizontal shear requirements.

**A.1.13  
PRETENSIONED  
ANCHORAGE ZONE**

[STD Art. 9.22]

**A.1.13.1  
Minimum Vertical  
Reinforcement**

In a pretensioned girder, vertical stirrups acting at a unit stress of 20,000 psi to resist at least 4 percent of the total pretensioning force must be placed within the distance of  $d/4$  of the girder end.

[STD Art. 9.22.1]

Minimum vertical stirrups at the each end of the girder:

$P_s$  = Prestressing force before initial losses have occurred, kips  
= (number of strands)(area of strand)(initial prestress)

Initial prestress,  $f_{si} = 0.75 f'_s$  [STD Art. 9.15.1]

where  $f'_s$  = Ultimate strength of prestressing strands = 270 ksi

$$f_{si} = 0.75(270) = 202.5 \text{ ksi}$$

$$P_s = 50(0.153)(202.5) = 1,549.13 \text{ kips}$$

Force to be resisted,  $F_s = 4$  percent of  $P_s = 0.04(1,549.13)$   
= 61.97 kips

Required area of stirrups to resist  $F_s$

$$A_v = \frac{F_s}{\text{Unit Stress in stirrups}}$$

Unit stress in stirrups = 20 ksi

$$A_v = \frac{61.97}{20} = 3.1 \text{ in.}^2$$

Distance available for placing the required area of stirrups =  $d/4$

where  $d$  is the distance from the extreme compressive fiber to centroid of pretensioned reinforcement = 48.76 in.

$$\frac{d}{4} = \frac{48.76}{4} = 12.19 \text{ in.}$$

Using six pairs of #5 bars @ 2 in. center-to-center spacing (within 12 in. from girder end) at each end of the girder:

$$A_v = 2(\text{area of each bar})(\text{number of bars}) \\ = 2(0.31)(6) = 3.72 \text{ in.}^2 > 3.1 \text{ in.}^2 \quad (\text{O.K.})$$

Therefore, provide 6 pairs of #5 bars @ 2 in. center-to-center spacing at each girder end.



### **A.1.13.2 Confinement Reinforcement**

STD Art. 9.22.2 specifies that the nominal reinforcement must be placed to enclose the prestressing steel in the bottom flange for a distance  $d$  from the end of the girder. [STD Art. 9.22.2]

where

$$\begin{aligned} d &= \text{Distance from the extreme compressive fiber to centroid of pretensioned reinforcement} \\ &= h_c - (y_b - e_x) = 62 - (24.75 - 11.51) = 48.76 \text{ in.} \end{aligned}$$

### **A.1.14 CAMBER AND DEFLECTIONS**

#### **A.1.14.1 Maximum Camber**

The Standard Specifications do not provide any guidelines for the determination camber of prestressed concrete members. The Hyperbolic Functions method proposed by Sinno and Furr (1970) for the calculation of maximum camber is used by TxDOT's prestressed concrete bridge design software, PSTRS14 (TxDOT 2004). The following steps illustrate the Hyperbolic Functions method for the estimation of maximum camber.

Step 1: The total prestressing force after initial prestress loss due to elastic shortening has occurred.

$$P = \frac{P_i}{\left(1 + pn + \frac{e_c^2 A_s n}{I}\right)} + \frac{M_D e_c A_s n}{I \left(1 + pn + \frac{e_c^2 A_s n}{I}\right)}$$

where:

$$\begin{aligned} P_i &= \text{Anchor force in prestressing steel} \\ &= (\text{number of strands})(\text{area of strand})(f_{si}) \end{aligned}$$

$$f_{si} = \text{Initial prestress before release} = 0.75 f'_s \quad [\text{STD Art. 9.15.1}]$$

$$f'_s = \text{Ultimate strength of prestressing strands} = 270 \text{ ksi}$$

$$f_{si} = 0.75(270) = 202.5 \text{ ksi}$$

$$P_i = 50(0.153)(202.5) = 1549.13 \text{ kips}$$

$$\begin{aligned} I &= \text{Moment of inertia of the non-composite precast girder} \\ &= 260403 \text{ in.}^4 \end{aligned}$$

$$e_c = \text{Eccentricity of prestressing strands at the midspan} \\ = 19.47 \text{ in.}$$

$$M_D = \text{Moment due to self-weight of the girder at midspan} \\ = 1209.98 \text{ k-ft.}$$

$$A_s = \text{Area of prestressing steel} \\ = (\text{number of strands})(\text{area of strand}) \\ = 50(0.153) = 7.65 \text{ in.}^2$$

$$p = A_s/A$$

$$A = \text{Area of girder cross-section} = 788.4 \text{ in.}^2$$

$$p = \frac{7.65}{788.4} = 0.0097$$

$$n = \text{Modular ratio between prestressing steel and the girder concrete at release} = E_s/E_{ci}$$

$$E_{ci} = \text{Modulus of elasticity of the girder concrete at release} \\ = 33(w_c)^{3/2} \sqrt{f'_{ci}} \quad [\text{STD Eq. 9-8}]$$

$$w_c = \text{Unit weight of concrete} = 150 \text{ pcf}$$

$$f'_{ci} = \text{Compressive strength of precast girder concrete at release} = 5455 \text{ psi}$$

$$E_{ci} = [33(150)^{3/2} \sqrt{5455}] \left( \frac{1}{1000} \right) = 4477.63 \text{ ksi}$$

$$E_s = \text{Modulus of elasticity of prestressing strands} \\ = 28,000 \text{ ksi}$$

$$n = 28,000/4477.63 = 6.25$$

$$\left( 1 + pn + \frac{e_c^2 A_s n}{I} \right) = 1 + (0.0097)(6.25) + \frac{(19.47^2)(7.65)(6.25)}{260,403} \\ = 1.130$$

$$P = \frac{1549.13}{1.130} + \frac{(1209.98)(12 \text{ in./ft.})(19.47)(7.65)(6.25)}{260,403(1.130)} \\ = 1370.91 + 45.93 = 1416.84 \text{ kips}$$

Initial prestress loss is defined as:

$$PL_i = \frac{P_i - P}{P} = \frac{1549.13 - 1416.84}{1549.13} = 0.0854 = 8.54\%$$

Note that the values obtained for initial prestress loss and effective initial prestress force using this methodology are comparable with the values obtained in Section A.1.7.3.5. The effective prestressing force after initial losses was found to be 1410.58 kips (comparable to 1416.84 kips), and the initial prestress loss was determined as 8.94 percent (comparable to 8.54 percent).

The stress in the concrete at the level of the centroid of the prestressing steel immediately after transfer is determined as follows.

$$f_{ci}^s = P \left( \frac{1}{A} + \frac{e_c^2}{I} \right) - f_c^s$$

where:

$$\begin{aligned} f_c^s &= \text{Concrete stress at the level of centroid of prestressing steel due to dead loads, ksi} \\ &= \frac{M_D e_c}{I} = \frac{(1209.98)(12 \text{ in./ft.})(19.47)}{260,403} = 1.0856 \text{ ksi} \end{aligned}$$

$$f_{ci}^s = 1416.84 \left( \frac{1}{788.4} + \frac{19.47^2}{260,403} \right) - 1.0856 = 2.774 \text{ ksi}$$

The ultimate time dependent prestress loss is dependent on the ultimate creep and shrinkage strains. As the creep strains vary with the concrete stress, the following steps are used to evaluate the concrete stresses and adjust the strains to arrive at the ultimate prestress loss. It is assumed that the creep strain is proportional to the concrete stress, and the shrinkage stress is independent of concrete stress (Sinno 1970).

Step 2: Initial estimate of total strain at steel level assuming constant sustained stress immediately after transfer

$$\epsilon_{c1}^s = \epsilon_{cr}^\infty f_{ci}^s + \epsilon_{sh}^\infty$$

where:

$$\epsilon_{cr}^\infty = \text{Ultimate unit creep strain} = 0.00034 \text{ in./in. [This value is prescribed by Sinno et al. (1970).]}$$

$\epsilon_{sh}^{\infty}$  = Ultimate unit shrinkage strain = 0.000175 in./in. [This value is prescribed by Sinno et al. (1970).]

$$\epsilon_{c1}^s = 0.00034(2.774) + 0.000175 = 0.001118 \text{ in./in.}$$

Step 3: The total strain obtained in Step 2 is adjusted by subtracting the elastic strain rebound as follows:

$$\epsilon_{c2}^s = \epsilon_{c1}^s - \epsilon_{c1}^s E_s \frac{A_s}{E_{ci}} \left( \frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\begin{aligned} \epsilon_{c2}^s &= 0.001118 - (0.001118)(28,000) \frac{7.65}{4477.63} \left( \frac{1}{788.4} + \frac{19.47^2}{260,403} \right) \\ &= 0.000972 \text{ in./in.} \end{aligned}$$

Step 4: The change in concrete stress at the level of centroid of prestressing steel is computed as follows:

$$\Delta f_c^s = \epsilon_{c2}^s E_s A_s \left( \frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\Delta f_c^s = (0.000972)(28,000)(7.65) \left( \frac{1}{788.4} + \frac{19.47^2}{260,403} \right) = 0.567 \text{ ksi}$$

Step 5: The total strain computed in Step 2 needs to be corrected for the change in the concrete stress due to creep and shrinkage strains.

$$\epsilon_{c4}^s = \epsilon_{cr}^{\infty} \left( f_{ci}^s - \frac{\Delta f_c^s}{2} \right) + \epsilon_{sh}^{\infty}$$

$$\epsilon_{c4}^s = 0.00034 \left( 2.774 - \frac{0.567}{2} \right) + 0.000175 = 0.00102 \text{ in./in.}$$

Step 6: The total strain obtained in Step 5 is adjusted by subtracting the elastic strain rebound as follows:

$$\epsilon_{c5}^s = \epsilon_{c4}^s - \epsilon_{c4}^s E_s \frac{A_s}{E_{ci}} \left( \frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\begin{aligned} \epsilon_{c5}^s &= 0.00102 - (0.00102)(28,000) \frac{7.65}{4477.63} \left( \frac{1}{788.4} + \frac{19.47^2}{260,403} \right) \\ &= 0.000887 \text{ in./in.} \end{aligned}$$

Sinno (1970) recommends stopping the updating of stresses and adjustment process after Step 6. However, as the difference between the strains obtained in Steps 3 and 6 is not negligible, this process is carried on until the total strain value converges.

Step 7: The change in concrete stress at the level of centroid of prestressing steel is computed as follows:

$$\Delta f_{c1}^s = \epsilon_{c5}^s E_s A_s \left( \frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\Delta f_{c1}^s = (0.000887)(28,000)(7.65) \left( \frac{1}{788.4} + \frac{19.47^2}{260,403} \right) = 0.5176 \text{ ksi}$$

Step 8: The total strain computed in Step 5 needs to be corrected for the change in the concrete stress due to creep and shrinkage strains.

$$\epsilon_{c6}^s = \epsilon_{cr}^\infty \left( f_{ci}^s - \frac{\Delta f_{c1}^s}{2} \right) + \epsilon_{sh}^\infty$$

$$\epsilon_{c6}^s = 0.00034 \left( 2.774 - \frac{0.5176}{2} \right) + 0.000175 = 0.00103 \text{ in./in.}$$

Step 9: The total strain obtained in Step 8 is adjusted by subtracting the elastic strain rebound as follows

$$\epsilon_{c7}^s = \epsilon_{c6}^s - \epsilon_{c6}^s E_s \frac{A_s}{E_{ci}} \left( \frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\begin{aligned} \epsilon_{c7}^s &= 0.00103 - (0.00103)(28,000) \frac{7.65}{4477.63} \left( \frac{1}{788.4} + \frac{19.47^2}{260,403} \right) \\ &= 0.000896 \text{ in./in} \end{aligned}$$

The strains have sufficiently converged, and no more adjustments are needed.

Step 10: Computation of final prestress loss

Time dependent loss in prestress due to creep and shrinkage strains is given as:

$$PL^\infty = \frac{\epsilon_{c7}^s E_s A_s}{P_i} = \frac{0.000896(28,000)(7.65)}{1549.13} = 0.124 = 12.4\%$$

Total final prestress loss is the sum of initial prestress loss and the time dependent prestress loss expressed as follows:

$$PL = PL_i + PL^\infty$$

where:

$PL$  = Total final prestress loss percent.

$PL_i$  = Initial prestress loss percent = 8.54 percent

$PL^\infty$  = Time dependent prestress loss percent = 12.4 percent

$PL = 8.54 + 12.4 = 20.94$  percent (This value of final prestress loss is less than the one estimated in Section A.1.7.3.6. where the final prestress loss was estimated to be 25.24 percent)

Step 11: The initial deflection of the girder under self-weight is calculated using the elastic analysis as follows:

$$C_{DL} = \frac{5 w L^4}{384 E_{ci} I}$$

where:

$C_{DL}$  = Initial deflection of the girder under self-weight, ft.

$w$  = Self-weight of the girder = 0.821 kips/ft.

$L$  = Total girder length = 109.67 ft.

$E_{ci}$  = Modulus of elasticity of the girder concrete at release  
= 4477.63 ksi = 644,778.72 k/ft.<sup>2</sup>

$I$  = Moment of inertia of the non-composite precast girder  
= 260,403 in.<sup>4</sup> = 12.558 ft.<sup>4</sup>

$$C_{DL} = \frac{5(0.821)(109.67^4)}{384(644,778.72)(12.558)} = 0.191 \text{ ft.} = 2.29 \text{ in.}$$

Step 12: Initial camber due to prestress is calculated using the moment area method. The following expression is obtained from the M/EI diagram to compute the camber resulting from the initial prestress.

$$C_{pi} = \frac{M_{pi}}{E_{ci} I}$$

where:

$$M_{pi} = [0.5(P) (e_e) (0.5L)^2 + 0.5(P) (e_c - e_e) (0.67) (HD)^2 + 0.5P (e_c - e_e) (HD_{dis}) (0.5L + HD)] / (Eci)(I)$$

$P$  = Total prestressing force after initial prestress loss due to elastic shortening has occurred = 1416.84 kips

$HD$  = Hold-down distance from girder end  
= 49.404 ft. = 592.85 in. (see Figure A.1.7.3)

$HD_{dis}$  = Hold-down distance from the center of the girder span  
=  $0.5(109.67) - 49.404 = 5.431$  ft. = 65.17 in.

$e_e$  = Eccentricity of prestressing strands at girder end  
= 11.07 in.

$e_c$  = Eccentricity of prestressing strands at midspan  
= 19.47 in.

$L$  = Overall girder length = 109.67 ft. = 1316.04 in.

$$M_{pi} = \{0.5(1416.84)(11.07) [0.5(1316.04)]^2 + 0.5(1416.84)(19.47 - 11.07)(0.67)(592.85)^2 + 0.5(1416.84)(19.47 - 11.07)(65.17)[0.5(1316.04) + 592.85]\}$$

$$M_{pi} = 3.396 \times 10^9 + 1.401 \times 10^9 + 0.485 \times 10^9 = 5.282 \times 10^9$$

$$C_{pi} = \frac{5.282 \times 10^9}{(4477.63)(260,403)} = 4.53 \text{ in.} = 0.378 \text{ ft.}$$

Step 13: The initial camber,  $C_i$ , is the difference between the upward camber due to initial prestressing and the downward deflection due to self-weight of the girder.

$$C_i = C_{pi} - C_{DL} = 4.53 - 2.29 = 2.24 \text{ in.} = 0.187 \text{ ft.}$$

Step 14: The ultimate time-dependent camber is evaluated using the following expression.

$$\text{Ultimate camber } C_t = C_i (1 - PL^\infty) \frac{\varepsilon_{cr}^\infty \left( f_{ci}^s - \frac{\Delta f_{cl}^s}{2} \right) + \varepsilon_e^s}{\varepsilon_e^s}$$

where:

$$\varepsilon_e^s = \frac{f_{ci}^s}{E_{ci}} = \frac{2.774}{4477.63} = 0.000619 \text{ in./in.}$$

$$C_t = 2.24(1 - 0.124) \frac{0.00034 \left( 2.774 - \frac{0.5176}{2} \right) + 0.000619}{0.000619}$$

$$C_t = 4.673 \text{ in.} = 0.389 \text{ ft. } \uparrow$$

#### **A.1.14.2 Deflection Due to Slab Weight**

The deflection due to the slab weight is calculated using an elastic analysis as follows.

Deflection of the girder at midspan

$$\Delta_{slab1} = \frac{5 w_s L^4}{384 E_c I}$$

where:

$$w_s = \text{Weight of the slab} = 0.80 \text{ kips/ft.}$$

$$\begin{aligned} E_c &= \text{Modulus of elasticity of girder concrete at service} \\ &= 33(w_c)^{3/2} \sqrt{f'_c} \\ &= 33(150)^{1.5} \sqrt{5582.5} \left( \frac{1}{1000} \right) = 4529.66 \text{ ksi} \end{aligned}$$

$$\begin{aligned} I &= \text{Moment of inertia of the non-composite girder section} \\ &= 260,403 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} L &= \text{Design span length of girder (center-to-center bearing)} \\ &= 108.583 \text{ ft.} \end{aligned}$$

$$\begin{aligned} \Delta_{slab1} &= \frac{5 \left( \frac{0.80}{12 \text{ in./ft.}} \right) [(108.583)(12 \text{ in./ft.})]^4}{384(4529.66)(260,403)} \\ &= 2.12 \text{ in.} = 0.177 \text{ ft. } \downarrow \end{aligned}$$



Deflection at quarter span due to slab weight

$$\Delta_{slab2} = \frac{57 w_s L^4}{6144 E_c I}$$

$$\Delta_{slab2} = \frac{57 \left( \frac{0.80}{12 \text{ in./ft.}} \right) [(108.583)(12 \text{ in./ft.})]^4}{6144(4529.66)(260,403)}$$

$$= 1.511 \text{ in.} = 0.126 \text{ ft.} \downarrow$$

**A.1.14.3**  
**Deflections due to**  
**Superimposed**  
**Dead Loads**

Deflection due to barrier weight at midspan

$$\Delta_{barr1} = \frac{5 w_{barr} L^4}{384 E_c I_c}$$

where:

$w_{barr}$  = Weight of the barrier = 0.109 kips/ft.

$I_c$  = Moment of inertia of composite section = 657,658.4 in<sup>4</sup>

$$\Delta_{barr1} = \frac{5 \left( \frac{0.109}{12 \text{ in./ft.}} \right) [(108.583)(12 \text{ in./ft.})]^4}{384(4529.66)(657,658.4)}$$

$$= 0.114 \text{ in.} = 0.0095 \text{ ft.} \downarrow$$

Deflection at quarter span due to barrier weight

$$\Delta_{barr2} = \frac{57 w_{barr} L^4}{6144 E_c I}$$

$$\Delta_{barr2} = \frac{57 \left( \frac{0.109}{12 \text{ in./ft.}} \right) [(108.583)(12 \text{ in./ft.})]^4}{6144(4529.66)(657,658.4)}$$

$$= 0.0815 \text{ in.} = 0.0068 \text{ ft.} \downarrow$$

Deflection due to wearing surface weight at midspan

$$\Delta_{ws1} = \frac{5 w_{ws} L^4}{384 E_c I_c}$$

where

$w_{ws}$  = Weight of wearing surface = 0.128 kips/ft.

$$\Delta_{ws1} = \frac{5 \left( \frac{0.128}{12 \text{ in./ft.}} \right) [(108.583)(12 \text{ in./ft.})]^4}{384(4529.66)(657,658.4)}$$

$$= 0.134 \text{ in.} = 0.011 \text{ ft.} \downarrow$$

Deflection at quarter span due to wearing surface

$$\Delta_{ws2} = \frac{57 w_{ws} L^4}{6144 E_c I}$$

$$\Delta_{ws2} = \frac{57 \left( \frac{0.128}{12 \text{ in./ft.}} \right) [(108.583)(12 \text{ in./ft.})]^4}{6144(4529.66)(657,658.4)}$$

$$= 0.096 \text{ in.} = 0.008 \text{ ft.} \downarrow$$

#### **A.1.14.4** **Total Deflection Due** **to Dead Loads**

The total deflection at midspan due to slab weight and superimposed loads is:

$$\Delta_{T1} = \Delta_{slab1} + \Delta_{barr1} + \Delta_{ws1}$$

$$= 0.177 + 0.0095 + 0.011 = 0.1975 \text{ ft.} \downarrow$$

The total deflection at quarter span due to slab weight and superimposed loads is:

$$\Delta_{T2} = \Delta_{slab2} + \Delta_{barr2} + \Delta_{ws2}$$

$$= 0.126 + 0.0068 + 0.008 = 0.1408 \text{ ft.} \downarrow$$

The deflections due to live loads are not calculated in this example as they are not a design factor for TxDOT bridges.

**A.1.15**  
**COMPARISON OF**  
**RESULTS FROM**  
**DETAILED DESIGN**  
**AND PSTRS14**

The prestressed concrete bridge girder design program, PSTRS14 (TxDOT 2004), is used by TxDOT for bridge design. The PSTRS14 program was run with same parameters as used in this detailed design, and the results of the detailed example and PSTRS14 program are compared in Table A.1.15.1.

*Table A.1.15.1. Comparison of the Results from PSTRS14 Program with Detailed Design Example.*

| Parameter  |              | PSTRS 14 Result | Detailed Design Result | Percent Difference |
|--|--------------|-----------------|------------------------|--------------------|
| Live Load Distribution Factor  |              | 0.727           | 0.727                  | 0.00               |
| Initial Prestress Loss   |              | 8.93%           | 8.94%                  | -0.11              |
| Final Prestress Loss   |              | 25.23%          | 25.24%                 | -0.04              |
| <i>Girder Stresses at Transfer</i>                                   |              |                 |                        |                    |
| At Girder End  | Top Fiber    | 35 psi          | 35 psi                 | 0.00               |
|  | Bottom Fiber | 3274 psi        | 3273 psi               | 0.03               |
| At Transfer Length Section   | Top Fiber    | Not Calculated  | 104 psi                | -                  |
|  | Bottom Fiber | Not calculated  | 3215 psi               | -                  |
| At Hold-Down   | Top Fiber    | 319 psi         | 351 psi                | -10.03             |
|  | Bottom Fiber | 3034 psi        | 3005 psi               | 1.00               |
| At Midspan   | Top Fiber    | 335 psi         | 368 psi                | -9.85              |
|  | Bottom Fiber | 3020 psi        | 2991 psi               | 0.96               |
| <i>Girder Stresses at Service</i>                                    |              |                 |                        |                    |
| At Girder End  | Top Fiber    | 29 psi          | Not calculated         | -                  |
|  | Bottom Fiber | 2688 psi        | Not calculated         | -                  |
| At Midspan   | Top Fiber    | 2563 psi        | 2562 psi               | 0.04               |
|  | Bottom Fiber | -414 psi        | -412 psi               | 0.48               |
| Slab Top Fiber Stress  |              | Not calculated  | 658 psi                | -                  |
| Required Concrete strength at Transfer                               |              | 5457 psi        | 5455 psi               | 0.04               |
| Required Concrete strength at Service                                |              | 5585 psi        | 5582.5 psi             | 0.04               |
| Total Number of Strands  |              | 50              | 50                     | 0.00               |
| Number of Harped Strands   |              | 10              | 10                     | 0.00               |
| Ultimate Flexural Moment Required                                    |              | 6771 k-ft.      | 6769.37 k-ft.          | 0.02               |
| Ultimate Moment Provided   |              | 8805 k-ft       | 8936.56 k-ft.          | -1.50              |
| Shear Stirrup Spacing at the Critical Section: double legged #4 bars |              | 21.4 in.        | 22 in.                 | -2.80              |
| Maximum Camber   |              | 0.306 ft.       | 0.389 ft.              | -27.12             |
| <i>Deflections</i>   |              |                 |                        |                    |
| Slab Weight  | Midspan      | -0.1601 ft.     | 0.1770 ft.             | -11.00             |
|  | Quarter Span | -0.1141 ft.     | 0.1260 ft.             | -10.00             |
| Barrier Weight   | Midspan      | -0.0096 ft.     | 0.0095 ft.             | 1.04               |
|  | Quarter Span | -0.0069 ft.     | 0.0068 ft.             | 1.45               |
| Wearing Surface Weight   | Midspan      | -0.0082 ft.     | 0.0110 ft.             | -34.10             |
|  | Quarter Span | -0.0058 ft.     | 0.0080 ft.             | -37.60             |

Except for a few differences, the results from the detailed design are in good agreement with the PSTRS 14 (TxDOT 2004) results. The causes for the differences in the results are discussed as follows.

1. *Girder stresses at transfer*: The detailed design example uses the overall girder length of 109'-8" for evaluating the stresses at transfer at the midspan section and hold-down point locations. The PSTRS 14 uses the design span length of 108'-7" for this calculation. This causes a difference in the stresses at transfer at hold-down point locations and midspan. The use of full girder length for stress calculations at transfer conditions seems to be appropriate as the girder rests on the ground, and the resulting moment is due to the self-weight of the overall girder.
2. *Maximum Camber*: The difference in the maximum camber results from detailed design and PSTRS 14 (TxDOT 2001) is occurring due to two reasons.
  - a. The detailed design example uses the overall girder length for the calculation of initial camber whereas, the PSTRS 14 program uses the design span length.
  - b. The updated composite section properties, based on the modular ratio between slab and actual girder concrete strengths are used for the camber calculations in the detailed design. However, the PSTRS 14 program does not update the composite section properties.
3. *Deflections*: The difference in the deflections is occurring due to the use of updated section properties and elastic modulus of concrete in the detailed design, based on the optimized concrete strength. However, the PSTRS 14 program does not update the composite section properties and uses the elastic modulus of concrete based on the initial input.

## **Appendix A.2**

### **Detailed Examples for Interior AASHTO Type IV Prestressed Concrete Bridge Girder Design using AASHTO LRFD Specifications**

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## A.2 Interior AASHTO Type IV Prestressed Concrete Bridge Girder Design using AASHTO LRFD Specifications

### A.2.1 INTRODUCTION

Following is a detailed example showing sample calculations for the design of a typical interior AASHTO Type IV prestressed concrete girder supporting a single span bridge. The design is based on the *AASHTO LRFD Bridge Design Specifications, 3<sup>rd</sup> Edition, 2004* (AASHTO 2004). The recommendations provided by the TxDOT Bridge Design Manual (TxDOT 2001) are considered in the design. The number of strands and concrete strength at release and at service are optimized using the TxDOT methodology.

### A.2.2 DESIGN PARAMETERS

The bridge considered for this design example has a span length of 110 ft. (c/c pier distance), a total width of 46 ft. and total roadway width of 44 ft. The bridge superstructure consists of six AASHTO Type IV girders spaced 8 ft. center-to-center, designed to act compositely with an 8 in. thick cast-in-place (CIP) concrete deck. The wearing surface thickness is 1.5 in., which includes the thickness of any future wearing surface. T501 type rails are considered in the design. HL-93 is the design live load. A relative humidity (RH) of 60 percent is considered in the design, and the skew angle is 0 degrees. The bridge cross-section is shown in Figure A.2.2.1.

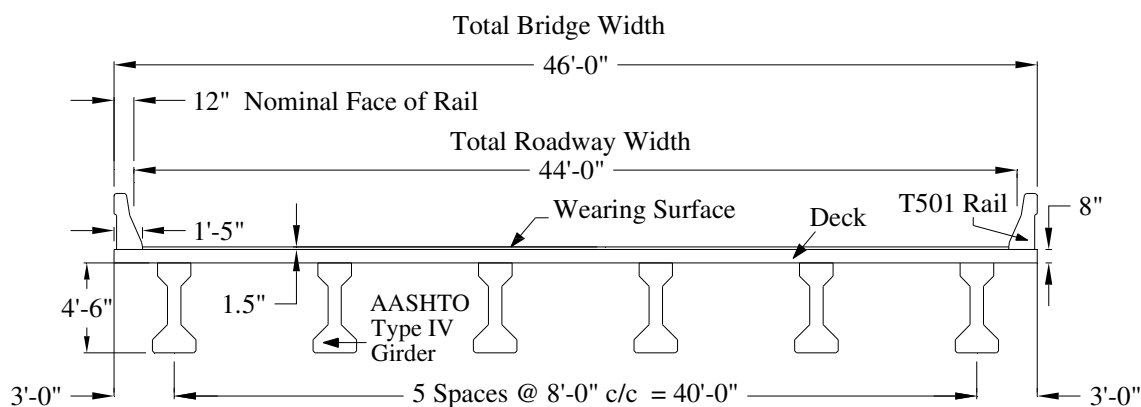


Figure A.2.2.1. Bridge Cross-Section Details.

The following calculations for design span length and the overall girder length are based on Figure A.2.2.2.

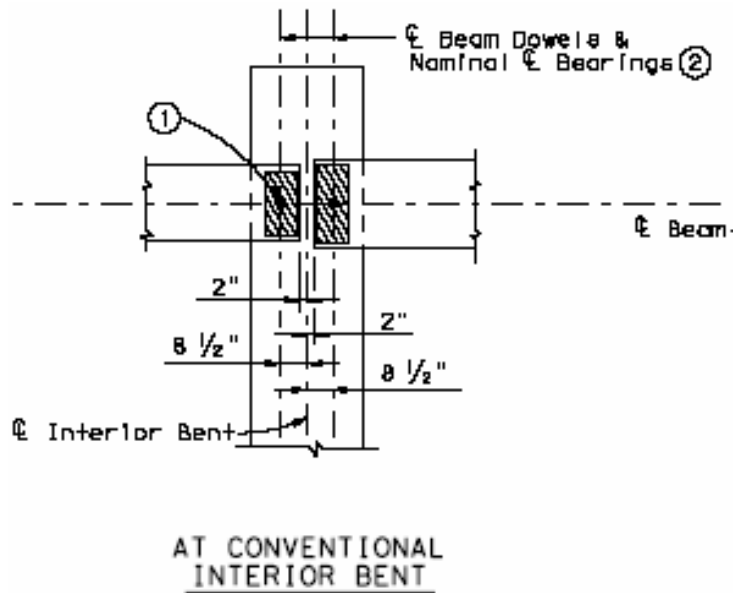


Figure A.2.2.2. Girder End Details  
(TxDOT Standard Drawing 2001).

Span Length (c/c piers) = 110'-0"

From Figure A.2.2.2

Overall girder length = 110'-0" - 2(2") = 109'-8" = 109.67 ft.

Design Span = 110'-0" - 2(8.5") = 108'-7" = 108.583 ft. (c/c of bearing)

### A.2.3 MATERIAL PROPERTIES

Cast-in-place slab:

Thickness,  $t_s = 8.0$  in.

Concrete strength at 28 days,  $f'_c = 4000$  psi

Thickness of asphalt wearing surface (including any future wearing surface),  $t_w = 1.5$  in.

Unit weight of concrete,  $w_c = 150$  pcf

Precast girders: AASHTO Type IV

Concrete strength at release,  $f'_{ci} = 4000$  psi (This value is taken as an initial estimate and will be finalized based on optimum design.)

Concrete strength at 28 days,  $f'_c = 5000$  psi (This value is taken as initial estimate and will be finalized based on optimum design.)

Concrete unit weight,  $w_c = 150$  pcf

Pretensioning strands: 0.5 in. diameter, seven wire low relaxation

Area of one strand =  $0.153 \text{ in.}^2$

Ultimate stress,  $f_{pu} = 270,000$  psi

Yield strength,  $f_{py} = 0.9f_{pu} = 243,000$  psi  
[LRFD Table 5.4.4.1-1]

Stress limits for prestressing strands: [LRFD Table 5.9.3-1]

Before transfer,  $f_{pi} \leq 0.75 f_{pu} = 202,500$  psi

At service limit state (after all losses)  
 $f_{pe} \leq 0.80 f_{py} = 194,400$  psi

Modulus of Elasticity,  $E_p = 28,500$  ksi [LRFD Art. 5.4.4.2]

Nonprestressed reinforcement:

Yield strength,  $f_y = 60,000$  psi

Modulus of Elasticity,  $E_s = 29,000$  ksi [LRFD Art. 5.4.3.2]

Unit weight of asphalt wearing surface = 140 pcf  
[TxDOT recommendation]

T501 type barrier weight = 326 plf /side

**A.2.4**  
**CROSS-SECTION**  
**PROPERTIES FOR A**  
**TYPICAL INTERIOR**  
**GIRDER**  
**A.2.4.1**  
**Non-Composite**  
**Section**

The section properties of an AASHTO Type IV girder as described in the TxDOT Bridge Design Manual (TxDOT 2001) are provided in Table A.2.4.1. The section geometry and strand pattern are shown in Figure A.2.4.1.

*Table A.2.4.1. Section Properties of AASHTO Type IV Girder  
[Adapted from TxDOT Bridge Design Manual (TxDOT 2001)].*

| $y_t$ | $y_b$ | Area             | $I$              | Wt./lf |
|-------|-------|------------------|------------------|--------|
| in.   | in.   | in. <sup>2</sup> | in. <sup>4</sup> | lbs    |
| 29.25 | 24.75 | 788.4            | 260,403          | 821    |

where:

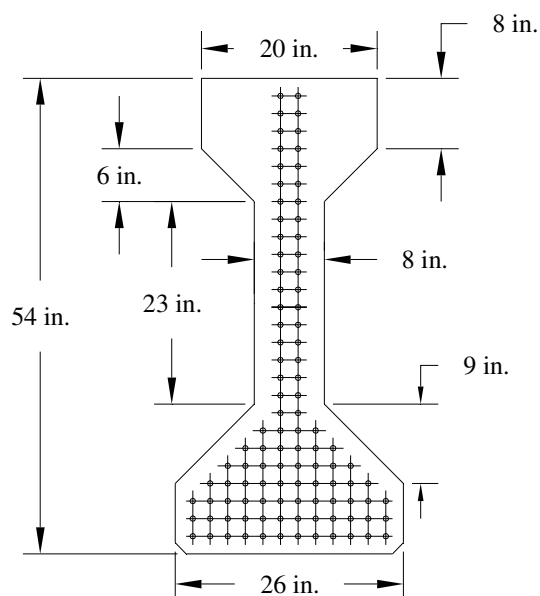
$I$  = Moment of inertia about the centroid of the non-composite precast girder = 260,403 in.<sup>4</sup>

$y_b$  = Distance from centroid to the extreme bottom fiber of the non-composite precast girder = 24.75 in.

$y_t$  = Distance from centroid to the extreme top fiber of the non-composite precast girder = 29.25 in.

$S_b$  = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder, in.<sup>3</sup>  
 $= I/y_b = 260,403/24.75 = 10,521.33$  in.<sup>3</sup>

$S_t$  = Section modulus referenced to the extreme top fiber of the non-composite precast girder, in.<sup>3</sup>  
 $= I/y_t = 260,403/29.25 = 8902.67$  in.<sup>3</sup>



*Figure A.2.4.1. Section Geometry and Strand Pattern for AASHTO Type IV Girder [Adapted from TxDOT Bridge Design Manual (TxDOT 2001)].*

**A.2.4.2**  
**Composite Section**  
**A.2.4.2.1**  
**Effective Flange**  
**Width**

[LRFD Art. 4.6.2.6.1]

The effective flange width is lesser of:

$$\frac{1}{4} \text{ span length of girder: } \frac{108.583(12 \text{ in./ft.})}{4} = 325.75 \text{ in.}$$

$$12 \times (\text{effective slab thickness}) + (\text{greater of web thickness or } \frac{1}{2} \text{ girder top flange width}): 12(8) + 0.5(20) = 106 \text{ in.}$$

$$(0.5 \times (\text{girder top flange width}) = 10 \text{ in.} > \text{web thickness} = 8 \text{ in.})$$

$$\text{Average spacing of adjacent girders: } (8 \text{ ft.})(12 \text{ in./ft.}) = 96 \text{ in.}$$

(controls)

$$\text{Effective flange width} = 96 \text{ in.}$$

**A.2.4.2.2**  
**Modular Ratio**  
**between Slab and**  
**Girder Concrete**

Following the TxDOT Bridge Design Manual (TxDOT 2001) recommendation (pg. 7-85), the modular ratio between the slab and girder concrete is taken as 1. This assumption is used for service load design calculations. For the flexural strength limit design, shear design, and deflection calculations, the actual modular ratio based on optimized concrete strengths is used. The composite section is shown in Figure A.2.4.2 and the composite section properties are presented in Table A.2.4.2.

$$n = \left( \frac{E_c \text{ for slab}}{E_c \text{ for girder}} \right) = 1$$

where  $n$  is the modular ratio between slab and girder concrete, and  $E_c$  is the elastic modulus of concrete.

**A.2.4.2.3**  
**Transformed Section**  
**Properties**

$$\text{Transformed flange width} = n \times (\text{effective flange width})$$

$$= (1)(96) = 96 \text{ in.}$$

$$\text{Transformed Flange Area} = n \times (\text{effective flange width})(t_s)$$

$$= (1)(96)(8) = 768 \text{ in.}^2$$

*Table A.2.4.2. Properties of Composite Section.*

|          | Transformed Area<br>$A \text{ (in.}^2\text{)}$ | $y_b$<br>in. | $A y_b$  | $A(y_{bc} - y_b)^2$ | $I$<br>in. <sup>4</sup> | $I + A(y_{bc} - y_b)^2$<br>in. <sup>4</sup> |
|----------|--|--------------|----------|---------------------|-------------------------|---|
| Girder   | 788.4  | 24.75        | 19,512.9 | 212,231.53          | 260,403.0               | 472,634.5                                   |
| Slab     | 768.0  | 58.00        | 44,544.0 | 217,868.93          | 4096.0                  | 221,964.9                                   |
| $\Sigma$ | 1556.4   |              | 64,056.9 |                     |                         | 694,599.5                                   |

$$A_c = \text{Total area of composite section} = 1556.4 \text{ in.}^2$$

$$h_c = \text{Total height of composite section} = 54 + 8 = 62 \text{ in.}$$

$$I_c = \text{Moment of inertia about the centroid of the composite section} = 694,599.5 \text{ in.}^4$$

$$y_{bc} = \text{Distance from the centroid of the composite section to extreme bottom fiber of the precast girder, in.} \\ = 64,056.9/1556.4 = 41.157 \text{ in.}$$

$$y_{tg} = \text{Distance from the centroid of the composite section to extreme top fiber of the precast girder, in.} \\ = 54 - 41.157 = 12.843 \text{ in.}$$

$$y_{tc} = \text{Distance from the centroid of the composite section to extreme top fiber of the slab} = 62 - 41.157 = 20.843 \text{ in.}$$

$$S_{bc} = \text{Section modulus of composite section referenced to the extreme bottom fiber of the precast girder, in.}^3 \\ = I_c/y_{bc} = 694,599.5/41.157 = 16,876.83 \text{ in.}^3$$

$$S_{tg} = \text{Section modulus of composite section referenced to the top fiber of the precast girder, in.}^3 \\ = I_c/y_{tg} = 694,599.5/12.843 = 54,083.9 \text{ in.}^3$$

$$S_{tc} = \text{Section modulus of composite section referenced to the top fiber of the slab, in.}^3 \\ = I_c/y_{tc} = 694,599.5/20.843 = 33,325.31 \text{ in.}^3$$

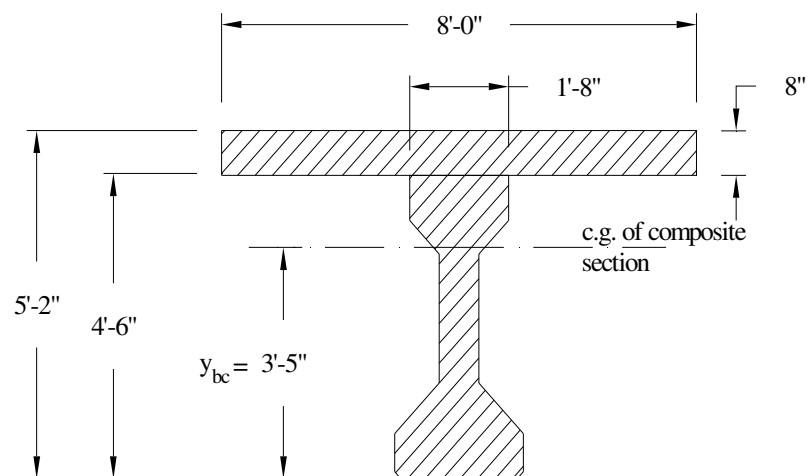


Figure A.2.4.3. Composite Section.

## **A.2.5 SHEAR FORCES AND BENDING MOMENTS**

The self-weight of the girder and the weight of the slab act on the non-composite simple span structure, while the weight of the barriers, future wearing surface, live load, and dynamic load act on the composite simple span structure.

### **A.2.5.1 Shear Forces and Bending Moments due to Dead Loads**

#### **A.2.5.1.1 Dead Loads**

[LRFD Art. 3.3.2]

Dead loads acting on the non-composite structure:

Self-weight of the girder = 0.821 kip/ft.

[TxDOT Bridge Design Manual (TxDOT 2001)]

Weight of cast-in-place deck on each interior girder

$$= (0.150 \text{ kcf}) \left( \frac{8 \text{ in.}}{12 \text{ in./ft.}} \right) (8 \text{ ft.}) = 0.800 \text{ kips/ft.}$$

Total dead load on non-composite section

$$= 0.821 + 0.800 = 1.621 \text{ kips/ft.}$$

#### **A.2.5.1.2 Superimposed Dead Loads**

The superimposed dead loads placed on the bridge, including loads from railing and wearing surface, can be distributed uniformly among all girders given the following conditions are met.

[LRFD Art. 4.6.2.2.1]

1. Width of deck is constant (O.K.)
2. Number of girders,  $N_b$ , is not less than four  
Number of girders in present case,  $N_b = 6$  (O.K.)
3. Girders are parallel and have approximately the same stiffness (O.K.)
4. The roadway part of the overhang,  $d_e \leq 3.0$  ft.  
where  $d_e$  is the distance from the exterior web of the exterior girder to the interior edge of the curb or traffic barrier, ft. (see Figure A.2.5.1)

$$\begin{aligned} d_e &= (\text{overhang distance from the center of the exterior girder to the bridge end}) - 0.5 \times (\text{web width}) - (\text{width of barrier}) \\ &= 3.0 - 0.33 - 1.0 = 1.67 \text{ ft.} < 3.0 \text{ ft.} \quad (\text{O.K.}) \end{aligned}$$

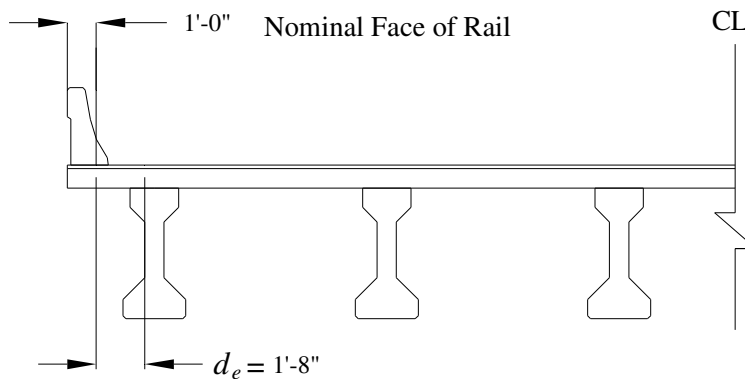


Figure A.2.5.1. Illustration of  $d_e$  Calculation.

5. Curvature in plan is less than  $4^0$  (curvature =  $0^0$ ) (O.K.)
6. Cross-section of the bridge is consistent with one of the cross-sections given in LRFD Table 4.6.2.2.1-1  
Precast concrete I sections are specified as Type k (O.K.)

Because all of the above criteria are satisfied, the barrier and wearing surface loads are equally distributed among the six girders.

Weight of T501 rails or barriers on each girder

$$= 2 \left( \frac{326 \text{ plf} / 1000}{6 \text{ girders}} \right) = 0.109 \text{ kips/ft./girder}$$

Weight of 1.5 in. wearing surface

$$= (0.140 \text{ kcf}) \left( \frac{1.5 \text{ in.}}{12 \text{ in./ft.}} \right) = 0.0175 \text{ kips/ft.}$$

This load is applied over the entire clear roadway width of 44'-0"

Weight of wearing surface on each girder

$$= \frac{(0.0175 \text{ ksf})(44.0 \text{ ft.})}{6 \text{ girders}} = 0.128 \text{ kips/ft./girder}$$

Total superimposed dead load =  $0.109 + 0.128 = 0.237 \text{ kips/ft.}$

### A.2.5.1.3 Shear Forces and Bending Moments

Shear forces and bending moments for the girder due to dead loads, superimposed dead loads at every tenth of the design span, and at critical sections (hold-down point or harp point and critical section



for shear) are provided in this section. The bending moment ( $M$ ) and shear force ( $V$ ) due to uniform dead loads and uniform superimposed dead loads at any section at a distance  $x$  from the centerline of bearing are calculated using the following formulas, where the uniform load is denoted as  $w$ .

$$M = 0.5w x (L - x)$$

$$V = w(0.5L - x)$$

The distance of the critical section for shear from the support is calculated using an iterative process illustrated in the shear design section. As an initial estimate, the distance of the critical section for shear from the centerline of bearing is taken as:

$$(h_c/2) + 0.5(\text{bearing width}) = (62/2) + 0.5(7) = 34.5 \text{ in.} = 2.875 \text{ ft.}$$

As per the recommendations of the TxDOT Bridge Design Manual (Chap. 7, Sec. 21), the distance of the hold-down ( $HD$ ) point from the centerline of bearing is taken as the lesser of:

$$[0.5 \times (\text{span length}) - (\text{span length}/20)] \text{ or } [0.5 \times (\text{span length}) - 5 \text{ ft.}]$$

$$\frac{108.583}{2} - \frac{108.583}{20} = 48.862 \text{ ft. or } \frac{108.583}{2} - 5 = 49.29 \text{ ft.}$$

$$HD = 48.862 \text{ ft.}$$

The shear forces and bending moments due to dead loads and superimposed loads are shown in Tables A.2.5.1 and A.2.5.2, respectively.

Table A.2.5.1. Shear Forces due to Dead and Superimposed Dead Loads.

| Distance from Bearing Centerline<br>$x$ | Section<br>$x/L$ | Dead Loads    |             | Superimposed Dead Loads |                        |       | Total Dead Load |
|---|------------------|---------------|-------------|-------------------------|------------------------|-------|-----------------|
|   |                  | Girder Weight | Slab Weight | Barrier Weight          | Wearing Surface Weight | Total |                 |
| ft.                                     |                  | kips          | kips        | kips                    | kips                   | kips  | kips            |
| 0.000                                   | 0.000            | 44.57         | 43.43       | 5.92                    | 6.95                   | 12.87 | 100.87          |
| 2.875                                   | 0.026            | 42.21         | 41.13       | 5.60                    | 6.58                   | 12.19 | 95.53           |
| 10.858                                  | 0.100            | 35.66         | 34.75       | 4.73                    | 5.56                   | 10.29 | 80.70           |
| 21.717                                  | 0.200            | 26.74         | 26.06       | 3.55                    | 4.17                   | 7.72  | 60.52           |
| 32.575                                  | 0.300            | 17.83         | 17.37       | 2.37                    | 2.78                   | 5.15  | 40.35           |
| 43.433                                  | 0.400            | 8.91          | 8.69        | 1.18                    | 1.39                   | 2.57  | 20.17           |
| 48.862                                  | 0.450 ( $HD$ )   | 4.46          | 4.34        | 0.59                    | 0.69                   | 1.29  | 10.09           |
| 54.292                                  | 0.500            | 0.00          | 0.00        | 0.00                    | 0.00                   | 0.00  | 0.00            |

Table A.2.5.2. *Bending Moments due to Dead and Superimposed Dead Loads.*

| Distance from Bearing Centerline<br>$x$ | Section<br>$x/L$ | Dead Loads    |             | Superimposed Dead Loads |                        |        | Total Dead Load |
|---|------------------|---------------|-------------|-------------------------|------------------------|--------|-----------------|
|   |                  | Girder Weight | Slab Weight | Barrier Weight          | Wearing Surface Weight | Total  |                 |
| ft.                                     |                  | k-ft.         | k-ft.       | k-ft.                   | k-ft.                  | k-ft.  | k-ft.           |
| 0.000                                   | 0.000            | 0.00          | 0.00        | 0.00                    | 0.00                   | 0.00   | 0.00            |
| 2.875                                   | 0.026            | 124.76        | 121.56      | 16.56                   | 19.45                  | 36.01  | 282.33          |
| 10.858                                  | 0.100            | 435.59        | 424.45      | 57.83                   | 67.91                  | 125.74 | 985.78          |
| 21.717                                  | 0.200            | 774.38        | 754.58      | 102.81                  | 120.73                 | 223.54 | 1752.51         |
| 32.575                                  | 0.300            | 1016.38       | 990.38      | 134.94                  | 158.46                 | 293.40 | 2300.16         |
| 43.433                                  | 0.400            | 1161.58       | 1131.87     | 154.22                  | 181.10                 | 335.32 | 2628.76         |
| 48.862                                  | 0.450 (HD)       | 1197.87       | 1167.24     | 159.04                  | 186.76                 | 345.79 | 2710.90         |
| 54.292                                  | 0.500            | 1209.98       | 1179.03     | 160.64                  | 188.64                 | 349.29 | 2738.29         |

**A.2.5.2****Shear Forces and  
Bending Moments  
due to Live Load****A.2.5.2.1****Live Load**

[LRFD Art. 3.6.1.2]

The LRFD Specifications specify a significantly different live load as compared to the Standard Specifications. The LRFD design live load is designated as HL-93, which consists of a combination of:

- Design truck with dynamic allowance or design tandem with dynamic allowance, whichever produces greater moments and shears, and
- Design lane load without dynamic allowance.

[LRFD Art. 3.6.1.2.2]

The design truck is designated as HS 20-44 consisting of an 8 kip front axle and two 32 kip rear axles.

[LRFD Art. 3.6.1.2.3]

The design tandem consists of a pair of 25-kip axles spaced 4 ft. apart. However, for spans longer than 40 ft. the tandem loading does not govern, thus only the truck load is investigated in this example.

[LRFD Art. 3.6.1.2.4]

The lane load consists of a load of 0.64 klf uniformly distributed in the longitudinal direction.

**A.2.5.2.2**  
**Live Load Distribution**  
**Factors for a Typical**  
**Interior Girder**

The distribution factors specified by the LRFD Specifications have changed significantly as compared to the Standard Specifications, which specify  $S/11$  ( $S$  is the girder spacing) to be used as the distribution factor.

[LRFD Art. 4.6.2.2]

The bending moments and shear forces due to live load can be distributed to individual girders using simplified approximate distribution factors specified by the LRFD Specifications. However, the simplified live load distribution factors can be used only if the following conditions are met:

[LRFD Art. 4.6.2.2.1]

1. Width of deck is constant (O.K.)
2. Number of girders,  $N_b$ , is not less than four  
 Number of girders in present case,  $N_b = 6$  (O.K.)
3. Girders are parallel and have approximately the same stiffness (O.K.)
4. The roadway part of the overhang,  $d_e \leq 3.0$  ft.  
 where  $d_e$  is the distance from exterior web of the exterior girder to the interior edge of curb or traffic barrier, ft.  

$$d_e = (\text{overhang distance from the center of the exterior girder to the bridge end}) - 0.5 \times (\text{web width}) - (\text{width of barrier})$$

$$= 3.0 - 0.33 - 1.0 = 1.67 \text{ ft.} < 3.0 \text{ ft.} \quad (\text{O.K.})$$
5. Curvature in plan is less than  $4^0$  (curvature =  $0^0$ ) (O.K.)
6. Cross-section of the bridge is consistent with one of the cross-sections given in LRFD Table 4.6.2.2.1-1
7. Precast concrete I sections are specified as Type k (O.K.)

The number of design lanes is computed as follows:

Number of design lanes = Integer part of the ratio  $w/12$

where  $w$  is the clear roadway width between the curbs = 44 ft.

[LRFD Art. 3.6.1.1.1]

Number of design lanes = Integer part of  $(44/12) = 3$  lanes.

**A.2.5.2.2.1**  
**Distribution Factor for**  
**Bending Moment**

The approximate live load moment distribution factors for interior girders are specified by LRFD Table 4.6.2.2.2b-1. The distribution factors for type k (prestressed concrete I section) bridges can be used if the following additional requirements are satisfied:

$3.5 \leq S \leq 16$ , where  $S$  is the spacing between adjacent girders, ft.  
 $S = 8.0$  ft (O.K.)

$4.5 \leq t_s \leq 12$ , where  $t_s$  is the slab thickness, in.  
 $t_s = 8.0$  in (O.K.)

$20 \leq L \leq 240$ , where  $L$  is the design span length, ft.  
 $L = 108.583$  ft. (O.K.)

$N_b \geq 4$ , where  $N_b$  is the number of girders in the cross-section.  
 $N_b = 6$  (O.K.)

$10,000 \leq K_g \leq 7,000,000$ , where  $K_g$  is the longitudinal stiffness parameter, in.<sup>4</sup>

$$K_g = n(I + A e_g^2) \quad [\text{LRFD Art. 3.6.1.1.1}]$$

where:

$$\begin{aligned} n &= \text{Modular ratio between girder and slab concrete.} \\ &= \frac{E_c \text{ for girder concrete}}{E_c \text{ for deck concrete}} = 1 \end{aligned}$$

Note that this ratio is the inverse of the one defined for composite section properties in Section A.2.4.2.2.

$$\begin{aligned} A &= \text{Area of girder cross-section (non-composite section)} \\ &= 788.4 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} I &= \text{Moment of inertia about the centroid of the non-composite precast girder} = 260,403 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} e_g &= \text{Distance between centers of gravity of the girder and slab, in.} \\ &= (t_s/2 + y_t) = (8/2 + 29.25) = 33.25 \text{ in.} \end{aligned}$$

$$K_g = 1[260,403 + 788.4 (33.25)^2] = 1,132,028.5 \text{ in.}^4 \quad (\text{O.K.})$$

The approximate live load moment distribution factors for interior girders specified by the LRFD Specifications are applicable in this case as all the requirements are satisfied. LRFD Table 4.6.2.2.2b-1 specifies the distribution factor for all limit states except fatigue limit state for interior type k girders as follows:

For one design lane loaded:

$$DFM = 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12.0 L t_s^3}\right)^{0.1}$$

where:

$DFM$  = Live load moment distribution factor for interior girders.

$S$  = Spacing of adjacent girders = 8 ft.

$L$  = Design span length = 108.583 ft.

$t_s$  = Thickness of slab = 8 in.

$$DFM = 0.06 + \left(\frac{8}{14}\right)^{0.4} \left(\frac{8}{108.583}\right)^{0.3} \left(\frac{1,132,028.5}{12.0(108.583)(8)^3}\right)^{0.1}$$

$$DFM = 0.06 + (0.8)(0.457)(1.054) = 0.445 \text{ lanes/girder}$$

For two or more lanes loaded:

$$DFM = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12.0 L t_s^3}\right)^{0.1}$$

$$DFM = 0.075 + \left(\frac{8}{9.5}\right)^{0.6} \left(\frac{8}{108.583}\right)^{0.2} \left(\frac{1,132,028.5}{12.0(108.583)(8)^3}\right)^{0.1}$$

$$= 0.075 + (0.902)(0.593)(1.054) = 0.639 \text{ lanes/girder}$$

The greater of the above two distribution factors governs. Thus, the case of two or more lanes loaded controls.

$$DFM = 0.639 \text{ lanes/girder}$$

**A.2.5.2.2.2**  
**Skew Reduction for**  
**DFM**

LRFD Article 4.6.2.2.2e specifies a skew reduction for load distribution factors for moment in longitudinal beams on skewed supports. LRFD Table 4.6.2.2.2e-1 presents the skew reduction formulas for skewed type k bridges where the skew angle  $\theta$  is such that  $30^\circ \leq \theta \leq 60^\circ$ .

For type k bridges having a skew angle such that  $\theta < 30^\circ$ , the skew reduction factor is specified as 1.0. For type k bridges having a skew angle  $\theta > 60^\circ$ , the skew reduction is the same as for  $\theta = 60^\circ$ .

For the present design, the skew angle is  $0^\circ$ ; thus a skew reduction for the live load moment distribution factor is not required.

**A.2.5.2.2.3**  
**Distribution Factor for**  
**Shear Force**

The approximate live load shear distribution factors for interior girders are specified by LRFD Table 4.6.2.2.3a-1. The distribution factors for type k (prestressed concrete I section) bridges can be used if the following requirements are satisfied:

$3.5 \leq S \leq 16$ , where  $S$  is the spacing between adjacent girders, ft.  
 $S = 8.0$  ft. (O.K.)

$4.5 \leq t_s \leq 12$ , where  $t_s$  is the slab thickness, in.  
 $t_s = 8.0$  in (O.K.)

$20 \leq L \leq 240$ , where  $L$  is the design span length, ft.  
 $L = 108.583$  ft. (O.K.)

$N_b \geq 4$ , where  $N_b$  is the number of girders in the cross-section.  
 $N_b = 6$  (O.K.)

The approximate live load shear distribution factors for interior girders specified by the LRFD Specifications are applicable in this case as all the requirements are satisfied. Table 4.6.2.2.3a-1 specifies the distribution factor for all limit states for interior type k girders as follows.

For one design lane loaded:

$$DFV = 0.36 + \left( \frac{S}{25.0} \right)$$

where:

$DFV$  = Live load shear distribution factor for interior girders.

$S$  = Spacing of adjacent girders = 8 ft.

$$DFV = 0.36 + \left( \frac{8}{25.0} \right) = 0.680 \text{ lanes/girder}$$

For two or more lanes loaded:

$$DFV = 0.2 + \left( \frac{S}{12} \right) - \left( \frac{S}{35} \right)^2$$

$$DFV = 0.2 + \frac{8}{12} - \left( \frac{8}{35} \right)^2 = 0.814 \text{ lanes/girder}$$

The greater of the above two distribution factors governs. Thus, the case of two or more lanes loaded controls.

$$DFV = 0.814 \text{ lanes/girder}$$

The distribution factor for live load moments and shears for the same case using the Standard Specifications is 0.727 lanes/girder.

#### **A.2.5.2.2.4 Skew Correction for DFV**

LRFD Article 4.6.2.2.3c specifies that the skew correction factor shall be applied to the approximate load distribution factors for shear in the interior girders on skewed supports. LRFD Table 4.6.2.2.3c-1 provides the correction factor for load distribution factors for support shear of the obtuse corner of skewed type k bridges where the following conditions are satisfied:

$0^\circ \leq \theta \leq 60^\circ$ , where  $\theta$  is the skew angle.

$\theta = 0^\circ$  (O.K.)

$3.5 \leq S \leq 16$ , where  $S$  is the spacing between adjacent girders, ft.

$S = 8.0$  ft. (O.K.)

$20 \leq L \leq 240$ , where  $L$  is the design span length, ft.

$L = 108.583$  ft. (O.K.)

$N_b \geq 4$ , where  $N_b$  is the number of girders in the cross-section.

$N_b = 6$  (O.K.)

The correction factor for load distribution factors for support shear of the obtuse corner of skewed type k bridges is given as:

$$1.0 + 0.20 \left( \frac{12.0 L t_s^3}{K_g} \right)^{0.3} \tan \theta = 1.0 \text{ when } \theta = 0^\circ$$

For the present design, the skew angle is  $0^\circ$ ; thus the skew correction for the live load shear distribution factor is not required.

### **A.2.5.2.3 Dynamic Allowance**

The LRFD Specifications specify the dynamic load effects as a percentage of the static live load effects. LRFD Table 3.6.2.1-1 specifies the dynamic allowance to be taken as 33 percent of the static load effects for all limit states, except the fatigue limit state, and 15 percent for the fatigue limit state. The factor to be applied to the static load shall be taken as:

$$(1 + IM/100)$$

where

$IM$  = Dynamic load allowance, applied to truck load or tandem load only  
 = 33 for all limit states except the fatigue limit state  
 = 15 for fatigue limit state

The Standard Specifications specify the impact factor to be calculated using the following equation

$$I = \frac{50}{L + 125} < 30\%$$

The impact factor was calculated to be 21.4 percent for the Standard design example.

### **A.2.5.2.4 Shear Forces and Bending Moments A.2.5.2.4.1 Due to Truck Load**

The maximum shear forces ( $V$ ) and bending moments ( $M$ ) due to HS 20-44 truck loading for all limit states, except for the fatigue limit state, on a per-lane-basis are calculated using the following formulas given in the *PCI Design Manual* (PCI 2003).

Maximum bending moment due to HS 20-44 truck load

For  $x/L = 0 - 0.333$

$$M = \frac{72(x)[(L - x) - 9.33]}{L}$$

For  $x/L = 0.333 - 0.5$

$$M = \frac{72(x)[(L - x) - 4.67]}{L} - 112$$

Maximum shear force due to HS 20-44 truck load

For  $x/L = 0 - 0.5$

$$V = \frac{72[(L - x) - 9.33]}{L}$$

where

$x$  = Distance from the centerline of bearing to the section at which bending moment or shear force is calculated, ft.

$L$  = Design span length = 108.583 ft.



Distributed bending moment due to truck load including dynamic load allowance ( $M_{LT}$ ) is calculated as follows:

$$\begin{aligned} M_{LT} &= (\text{Moment per lane due to truck load})(DFM)(1+IM/100) \\ &= (M)(0.639)(1 + 33/100) \\ &= (M)(0.85) \end{aligned}$$

Distributed shear force due to truck load including dynamic load allowance ( $V_{LT}$ ) is calculated as follows:

$$\begin{aligned} V_{LT} &= (\text{Shear force per lane due to truck load})(DFV)(1+IM/100) \\ &= (V)(0.814)(1 + 33/100) \\ &= (V)(1.083) \end{aligned}$$

where:

$M$  = Maximum bending moment due to HS 20-44 truck load, k-ft.

$DFM$  = Live load moment distribution factor for interior girders

$IM$  = Dynamic load allowance, applied to truck load or tandem load only

$DFV$  = Live load shear distribution factor for interior girders

$V$  = Maximum shear force due to HS 20-44 truck load, kips

The maximum bending moments and shear forces due to an HS 20-44 truck load are calculated at every tenth of the span length and at the critical section for shear and the hold-down point location. The values are presented in Table A.2.5.2.

#### **A.2.5.2.4.1** **Due to Design Lane** **Load**

The maximum bending moments ( $M_L$ ) and shear forces ( $V_L$ ) due to a uniformly distributed lane load of 0.64 klf are calculated using the following formulas given by the *PCI Design Manual* (PCI 2003).

Maximum bending moment,  $M_L = 0.5(0.64)(x)(L - x)$

where:

$x$  = Distance from the centerline of bearing to the section at which the bending moment or shear force is calculated, ft.

$L$  = Design span length = 108.583 ft.

$$\text{Maximum shear force, } V_L = \frac{0.32(L - x)^2}{L} \text{ for } x \leq 0.5L$$

(Note that maximum shear force at a section is calculated at a section by placing the uniform load on the right of the section considered as shown in Figure A.2.5.1, given by the *PCI Design Manual* (PCI 2003). This method yields a slightly conservative estimate of the shear force as compared to the shear force at a section under uniform load placed on the entire span length.)

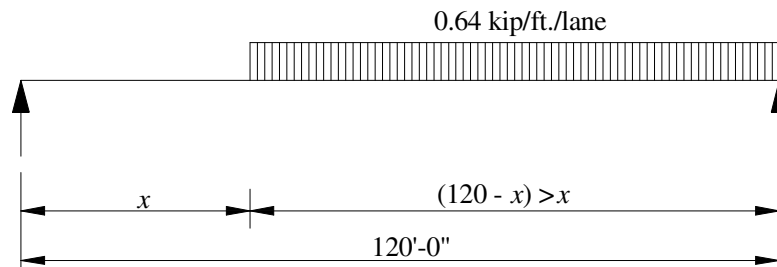


Figure A.2.5.1. Maximum Shear Force due to Lane Load.

Distributed bending moment due to lane load ( $M_{LL}$ ) is calculated as follows:

$$\begin{aligned} M_{LL} &= (\text{Moment per lane due to lane load})(DFM) \\ &= M_L (0.639) \end{aligned}$$

Distributed shear force due to lane load ( $V_{LL}$ ) is calculated as follows:

$$\begin{aligned} V_{LL} &= (\text{shear force per lane due to lane load})(DFV) \\ &= V_L (0.814) \end{aligned}$$

where:

$M_L$  = Maximum bending moment due to lane load, k-ft.

$DFM$  = Live load moment distribution factor for interior girders

$DFV$  = Live load shear distribution factor for interior girders

$V_L$  = Maximum shear force due to lane load, kips

The maximum bending moments and shear forces due to the lane load are calculated at every tenth of the span length and at the critical section for shear and the hold-down point location. The values are presented in Table A.2.5.2.

Table A.2.5.2. Shear Forces and Bending Moments due to Live Load.

| Distance<br>from<br>Bearing<br>Centerline<br>$x$<br>ft. | Section<br>$x/L$ | HS 20-44 Truck Loading      |         |                                     |          | Lane loading               |        |                          |          |
|---|------------------|-----------------------------|---------|-------------------------------------|----------|----------------------------|--------|--------------------------|----------|
|   |                  | Undistributed<br>Truck Load |         | Distributed Truck<br>+ Dynamic Load |          | Undistributed<br>Lane Load |        | Distributed Lane<br>Load |          |
|   |                  | Shear                       | Moment  | Shear                               | Moment   | Shear                      | Moment | Shear                    | Moment   |
|   |                  | $V$                         | $M$     | $V_{LT}$                            | $M_{LT}$ | $V_L$                      | $M_L$  | $V_{LL}$                 | $M_{LL}$ |
|   |                  | kips                        | k-ft.   | kips                                | k-ft.    | kips                       | k-ft.  | kips                     | k-ft.    |
| 0.000   | 0.000            | 65.81                       | 0.00    | 71.25                               | 0.00     | 34.75                      | 0.00   | 28.28                    | 0.00     |
| 2.875   | 0.026            | 63.91                       | 183.73  | 69.19                               | 156.15   | 32.93                      | 97.25  | 26.81                    | 62.14    |
| 10.858  | 0.100            | 58.61                       | 636.43  | 63.45                               | 540.88   | 28.14                      | 339.55 | 22.91                    | 216.97   |
| 21.717  | 0.200            | 51.41                       | 1116.54 | 55.66                               | 948.91   | 22.24                      | 603.67 | 18.10                    | 385.75   |
| 32.575  | 0.300            | 44.21                       | 1440.25 | 47.86                               | 1224.03  | 17.03                      | 792.31 | 13.86                    | 506.28   |
| 43.433  | 0.400            | 37.01                       | 1629.82 | 40.07                               | 1385.14  | 12.51                      | 905.49 | 10.18                    | 578.61   |
| 48.862  | 0.450 (HD)       | 33.41                       | 1671.64 | 36.17                               | 1420.68  | 10.51                      | 933.79 | 8.56                     | 596.69   |
| 54.292  | 0.500            | 29.81                       | 1674.37 | 32.27                               | 1423.00  | 8.69                       | 943.22 | 7.07                     | 602.72   |

**A.2.5.3****Load Combinations**

LRFD Art. 3.4.1 specifies load factors and load combinations. The total factored load effect is specified to be taken as:

$$Q = \sum \eta_i \gamma_i Q_i \quad [\text{LRFD Eq. 3.4.1-1}]$$

where

$Q$  = Factored force effects

$\gamma_i$  = Load factor, a statistically based multiplier applied to force effects specified by LRFD Table 3.4.1-1

$Q_i$  = Unfactored force effects

$\eta_i$  = Load modifier, a factor relating to ductility, redundancy and operational importance

=  $\eta_D \eta_R \eta_I \geq 0.95$ , for loads for which a maximum value of  $\gamma_i$  is appropriate [LRFD Eq. 1.3.2.1-2]

=  $\frac{1}{\eta_D \eta_R \eta_I} \leq 1.0$ , for loads for which a minimum value of  $\gamma_i$  is appropriate [LRFD Eq. 1.3.2.1-3]

$\eta_D$  = A factor relating to ductility

= 1.00 for all limit states except strength limit state

For the strength limit state:

$$\begin{aligned}\eta_D &\geq 1.05 \text{ for nonductile components and connections} \\ &= 1.00 \text{ for conventional design and details complying with the} \\ &\quad \text{LRFD Specifications} \\ &\geq 0.95 \text{ for components and connections for which additional} \\ &\quad \text{ductility-enhancing measures have been specified beyond} \\ &\quad \text{those required by the LRFD Specifications}\end{aligned}$$

$\eta_D = 1.00$  is used in this example for strength and service limit states as this design is considered to be conventional and complying with the LRFD Specifications.

$$\begin{aligned}\eta_R &= \text{A factor relating to redundancy} \\ &= 1.00 \text{ for all limit states except strength limit state}\end{aligned}$$

For strength limit state:

$$\begin{aligned}\eta_R &\geq 1.05 \text{ for nonredundant members} \\ &= 1.00 \text{ for conventional levels of redundancy} \\ &\geq 0.95 \text{ for exceptional levels of redundancy}\end{aligned}$$

$\eta_R = 1.00$  is used in this example for strength and service limit states as this design is considered to provide a conventional level of redundancy to the structure.

$$\begin{aligned}\eta_I &= \text{A factor relating to operational importance} \\ &= 1.00 \text{ for all limit states except strength limit state}\end{aligned}$$

For strength limit state:

$$\begin{aligned}\eta_I &\geq 1.05 \text{ for important bridges} \\ &= 1.00 \text{ for typical bridges} \\ &\geq 0.95 \text{ for relatively less important bridges}\end{aligned}$$

$\eta_I = 1.00$  is used in this example for strength and service limit states, as this example illustrates the design of a typical bridge.

$$\eta_i = \eta_D \eta_R \eta_I = 1.00 \text{ in present case} \quad [\text{LRFD Art. 1.3.2}]$$

The notations used in the following section are defined as follows:

$DC$  = Dead load of structural components and non-structural attachments

$DW$  = Dead load of wearing surface and utilities

$LL$  = Vehicular live load

$IM$  = Vehicular dynamic load allowance

This design example considers only the dead and vehicular live loads. The wind load and the extreme event loads, including earthquake and vehicle collision loads, are not included in the design, which is typical to the design of bridges in Texas. Various limit states and load combinations provided by LRFD Art. 3.4.1 are investigated, and the following limit states are found to be applicable in present case:

**Service I:** This limit state is used for normal operational use of a bridge. This limit state provides the general load combination for service limit state stress checks and applies to all conditions except Service III limit state. For prestressed concrete components, this load combination is used to check for compressive stresses. The load combination is presented as follows:

$$Q = 1.00(DC + DW) + 1.00(LL + IM) \quad [\text{LRFD Table 3.4.1-1}]$$

**Service III:** This limit state is a special load combination for service limit state stress checks that applies only to tension in prestressed concrete structures to control cracks. The load combination for this limit state is presented as follows:

$$Q = 1.00(DC + DW) + 0.80(LL + IM) \quad [\text{LRFD Table 3.4.1-1}]$$

**Strength I:** This limit state is the general load combination for strength limit state design relating to the normal vehicular use of the bridge without wind. The load combination is presented as follows:

[LRFD Table 3.4.1-1 and 2]

$$Q = \gamma_p(DC) + \gamma_p(DW) + 1.75(LL + IM)$$

$\gamma_p$  = Load factor for permanent loads provided in Table A.2.5.3.1

*Table A.2.5.3.1. Load Factors for Permanent Loads.*

| Type of Load   | Load Factor, $\gamma_p$ |         |
|--|-------------------------|---------|
|  | Maximum                 | Minimum |
| DC: Structural components and non-structural attachments | 1.25                    | 0.90    |
| DW: Wearing surface and utilities                        | 1.50                    | 0.65    |

The maximum and minimum load combinations for the Strength I limit state are presented as follows:

$$\text{Maximum } Q = 1.25(DC) + 1.50(DW) + 1.75(LL + IM)$$

$$\text{Minimum } Q = 0.90(DC) + 0.65(DW) + 1.75(LL + IM)$$

For simple span bridges, the maximum load factors produce maximum effects. However, minimum load factors are used for component dead loads ( $DC$ ) and wearing surface load ( $DW$ ) when dead load and wearing surface stresses are opposite to those of live load. In the present example, the maximum load factors are used to investigate the ultimate strength limit state.

### **A.2.6 ESTIMATION OF REQUIRED PRESTRESS**

The required number of strands is usually governed by concrete tensile stress at the bottom fiber of the girder at the midspan section. The load combination for Service III limit state is used to evaluate the bottom fiber stresses at the midspan section. The calculation for compressive stress in the top fiber of the girder at midspan section under service loads is also shown in the following section. The compressive stress is evaluated using the load combination for Service I limit state.

#### **A.2.6.1 Service Load Stresses at Midspan**

Tensile stress at the bottom fiber of the girder at midspan due to applied dead and live loads using load combination Service III

$$f_b = \frac{M_{DCN}}{S_b} + \frac{M_{DCC} + M_{DW} + 0.8(M_{LT} + M_{LL})}{S_{bc}}$$

Compressive stress at the top fiber of the girder at midspan due to applied dead and live loads using load combination Service I

$$f_t = \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW} + M_{LT} + M_{LL}}{S_{tg}}$$

where:

$f_b$  = Concrete stress at the bottom fiber of the girder, ksi

$f_t$  = Concrete stress at the top fiber of the girder, ksi

$M_{DCN}$  = Moment due to non-composite dead loads, k-ft.  
=  $M_g + M_s$

$M_g$  = Moment due to girder self-weight = 1209.98 k-ft.

$M_s$  = Moment due to slab weight = 1179.03 k-ft.

$M_{DCN} = 1209.98 + 1179.03 = 2389.01$  k-ft.

$$M_{DCC} = \text{Moment due to composite dead loads except wearing surface load, k-ft.}$$

$$= M_{barr}$$

$$M_{barr} = \text{Moment due to barrier weight} = 160.64 \text{ k-ft.}$$

$$M_{DCC} = 160.64 \text{ k-ft.}$$

$$M_{DW} = \text{Moment due to wearing surface load} = 188.64 \text{ k-ft.}$$

$$M_{LT} = \text{Distributed moment due to HS 20-44 truck load including dynamic load allowance} = 1423.00 \text{ k-ft.}$$

$$M_{LL} = \text{Distributed moment due to lane load} = 602.72 \text{ k-ft.}$$

$$S_b = \text{Section modulus referenced to the extreme bottom fiber of the non-composite precast girder} = 10,521.33 \text{ in.}^3$$

$$S_t = \text{Section modulus referenced to the extreme top fiber of the non-composite precast girder} = 8902.67 \text{ in.}^3$$

$$S_{bc} = \text{Section modulus of composite section referenced to the extreme bottom fiber of the precast girder}$$

$$= 16,876.83 \text{ in.}^3$$

$$S_{tg} = \text{Section modulus of composite section referenced to the top fiber of the precast girder} = 54,083.9 \text{ in.}^3$$

Substituting the bending moments and section modulus values, stresses at bottom fiber ( $f_b$ ) and top fiber ( $f_t$ ) of the girder at midspan section are:

$$f_b = \frac{(2389.01)(12 \text{ in./ft.})}{10,521.33} + \frac{[160.64 + 188.64 + 0.8(1423.00 + 602.72)](12 \text{ in./ft.})}{16,876.83}$$

$$= 2.725 + 1.400 = 4.125 \text{ ksi (As compared to 4.024 ksi for design using Standard Specifications)}$$

$$f_t = \frac{(2389.01)(12 \text{ in./ft.})}{8902.67} + \frac{[160.64 + 188.64 + 1423.00 + 602.72](12 \text{ in./ft.})}{54,083.9}$$

$$= 3.220 + 0.527 = 3.747 \text{ ksi (As compared to 3.626 ksi for design using Standard Specifications)}$$

The stresses in the top and bottom fibers of the girder at the hold-down point, midspan, and top fiber of the slab are calculated in a similar way as shown above and the results are summarized in Table A.2.6.1.

Table A.2.6.1. Summary of Stresses due to Applied Loads.

| Load  | Stresses in Girder       |                    |                   |                    | Stresses in Slab  |
|---|--------------------------|--------------------|-------------------|--------------------|-------------------|
|   | Stress at Hold-Down (HD) |                    | Stress at Midspan |                    | Stress at Midspan |
|   | Top Fiber (psi)          | Bottom Fiber (psi) | Top Fiber (psi)   | Bottom Fiber (psi) | Top Fiber (psi)   |
| Girder self-weight  | 1614.63                  | -1366.22           | 1630.94           | -1380.03           | -                 |
| Slab weight   | 1573.33                  | -1331.28           | 1589.22           | -1344.73           | -                 |
| Barrier weight  | 35.29                    | -113.08            | 35.64             | -114.22            | 57.84             |
| Wearing surface weight  | 41.44                    | -132.79            | 41.85             | -134.13            | 67.93             |
| Total dead load   | 3264.68                  | -2943.38           | 3297.66           | -2973.10           | 125.77            |
| HS 20-44 truck load (multiplied by 0.8 for bottom fiber stress calculation) | 315.22                   | -808.12            | 315.73            | -809.44            | 512.40            |
| Lane load (multiplied by 0.8 for bottom fiber stress calculation)           | 132.39                   | -339.41            | 133.73            | -342.84            | 217.03            |
| Total live load   | 447.61                   | -1147.54           | 449.46            | -1152.28           | 729.43            |
| Total load  | 3712.29                  | -4090.91           | 3747.12           | -4125.39           | 855.21            |

(Negative values indicate tensile stress)

### A.2.6.2 Allowable Stress Limit

LRFD Table 5.9.4.2.2-1 specifies the allowable tensile stress in fully prestressed concrete members. For members with bonded prestressing tendons that are subjected to not worse than moderate corrosion conditions (these corrosion conditions are assumed in this design), the allowable tensile stress at service limit state after losses is given as:

$$F_b = 0.19\sqrt{f'_c}$$

where

$f'_c$  = Compressive strength of girder concrete at service = 5.0 ksi

$F_b = 0.19\sqrt{5.0} = 0.4248$  ksi (As compared to allowable tensile stress of 0.4242 ksi for the Standard design).



### A.2.6.3 Required Number of Strands

Required precompressive stress in the bottom fiber after losses:

Bottom tensile stress – Allowable tensile stress at service =  $f_b - F_b$

$$f_{pb-reqd.} = 4.125 - 0.4248 = 3.700 \text{ ksi}$$

Assuming the eccentricity of the prestressing strands at midspan ( $e_c$ ) as the distance from the centroid of the girder to the bottom fiber of the girder (PSTRS 14 methodology, TxDOT 2004)

$$e_c = y_b = 24.75 \text{ in.}$$

Stress at the bottom fiber of the girder due to prestress after losses:

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe} e_c}{S_b}$$

where:

$P_{pe}$  = Effective prestressing force after all losses, kips

$A$  = Area of girder cross-section = 788.4 in.<sup>2</sup>

$S_b$  = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder = 10,521.33 in.<sup>3</sup>

Required prestressing force is calculated by substituting the corresponding values in the above equation as follows.

$$3.700 = \frac{P_{pe}}{788.4} + \frac{24.75 P_{pe}}{10,521.33}$$

Solving for  $P_{pe}$ ,

$$P_{pe} = 1021.89 \text{ kips}$$

Assuming final losses = 20 percent of initial prestress  $f_{pi}$   
(TxDOT 2001)

$$\text{Assumed final losses} = 0.2(202.5) = 40.5 \text{ ksi}$$

The prestress force required per strand after losses  
= (cross-sectional area of one strand) [ $f_{pi}$  – losses]  
= 0.153(202.5 – 40.5) = 24.78 kips

$$\text{Number of prestressing strands required} = 1021.89/24.78 = 41.24$$

Try 42 – 0.5 in. diameter, 270 ksi low relaxation strands as an initial trial.

Strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2 + 4 + 6) + 6(8)}{42} = 20.18 \text{ in.}$$

Available prestressing force

$$P_{pe} = 42(24.78) = 1040.76 \text{ kips}$$

Stress at bottom fiber of the girder due to prestress after losses:

$$\begin{aligned} f_b &= \frac{1040.76}{788.4} + \frac{1040.76(20.18)}{10,521.33} \\ &= 1.320 + 1.996 = 3.316 \text{ ksi} < f_{pb-reqd.} = 3.700 \text{ ksi} \quad (\text{N.G.}) \end{aligned}$$

Try 44 – 0.5 in. diameter, 270 ksi low relaxation strands as an initial trial.

Strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2 + 4 + 6) + 8(8)}{44} = 20.02 \text{ in.}$$

Available prestressing force

$$P_{pe} = 44(24.78) = 1090.32 \text{ kips}$$

Stress at bottom fiber of the girder due to prestress after losses:

$$\begin{aligned} f_b &= \frac{1090.32}{788.4} + \frac{1090.32(20.02)}{10,521.33} \\ &= 1.383 + 2.075 = 3.458 \text{ ksi} < f_{pb-reqd.} = 3.700 \text{ ksi} \quad (\text{N.G.}) \end{aligned}$$

Try 46 – 0.5 in. diameter, 270 ksi low relaxation strands as an initial trial.

Strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2 + 4 + 6) + 10(8)}{46} = 19.88 \text{ in.}$$

Available prestressing force

$$P_{pe} = 46(24.78) = 1139.88 \text{ kips}$$

Stress at bottom fiber of the girder due to prestress after losses:

$$\begin{aligned} f_b &= \frac{1139.88}{788.4} + \frac{1139.88(19.88)}{10,521.33} \\ &= 1.446 + 2.154 = 3.600 \text{ ksi} < f_{pb-reqd.} = 3.700 \text{ ksi} \quad (\text{N.G.}) \end{aligned}$$

Try 48 – 0.5 in. diameter, 270 ksi low relaxation strands as an initial trial.

Strand eccentricity at midspan after strand arrangement

$$e_c = 24.75 - \frac{12(2+4+6) + 10(8)+2(10)}{48} = 19.67 \text{ in.}$$

Available prestressing force

$$P_{pe} = 48(24.78) = 1189.44 \text{ kips}$$

Stress at bottom fiber of the girder due to prestress after losses:

$$f_b = \frac{1189.44}{788.4} + \frac{1189.44(19.67)}{10,521.33}$$

$$= 1.509 + 2.223 = 3.732 \text{ ksi} > f_{pb-reqd.} = 3.700 \text{ ksi} \quad (\text{O.K.})$$

Therefore, use 48 strands as a preliminary estimate for the number of strands. The strand arrangement is shown in Figure A.2.6.1.

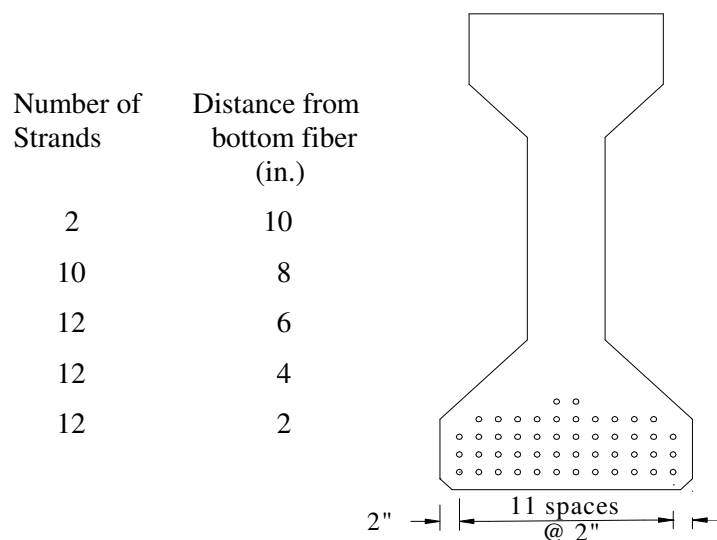


Figure A.2.6.1. Initial Strand Arrangement.

The distance from the center of gravity of the strands to the bottom fiber of the girder ( $y_{bs}$ ) is calculated as:

$$y_{bs} = y_b - e_c = 24.75 - 19.67 = 5.08 \text{ in.}$$

### A.2.7 PRESTRESS LOSSES

[LRFD Art. 5.9.5]

The LRFD Specifications specify formulas to determine the instantaneous losses. For time-dependent losses, two different options are provided. The first option is to use a lump-sum estimate of time-dependent losses given by LRFD Art. 5.9.5.3. The second option is to use refined estimates for time-dependent losses given by LRFD Art. 5.9.5.4. The refined estimates are used in this design as they yield more accuracy as compared to the lump-sum method.

The instantaneous loss of prestress is estimated using the following expression:

$$\Delta f_{pi} = (\Delta f_{pES} + \Delta f_{pRI})$$

The percent instantaneous loss is calculated using the following expression:

$$\% \Delta f_{pi} = \frac{100(\Delta f_{pES} + \Delta f_{pRI})}{f_{pj}}$$

TxDOT methodology was used for the evaluation of instantaneous prestress loss in the Standard design example given by the following expression.

$$\Delta f_{pi} = (ES + \frac{1}{2} CR_S)$$

where:

$\Delta f_{pi}$  = Instantaneous prestress loss, ksi

$\Delta f_{pES}$  = Prestress loss due to elastic shortening, ksi

$\Delta f_{pRI}$  = Prestress loss due to steel relaxation before transfer, ksi

$f_{pj}$  = Jacking stress in prestressing strands = 202.5 ksi

$ES$  = Prestress loss due to elastic shortening, ksi

$CR_S$  = Prestress loss due to steel relaxation at service, ksi

The time-dependent loss of prestress is estimated using the following expression:

$$\text{Time Dependent loss} = \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$$

where:

$\Delta f_{pSR}$  = Prestress loss due to concrete shrinkage, ksi

$\Delta f_{pCR}$  = Prestress loss due to concrete creep, ksi

$\Delta f_{pR2}$  = Prestress loss due to steel relaxation after transfer, ksi

The total prestress loss in prestressed concrete members prestressed in a single stage, relative to stress immediately before transfer is given as:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2} \quad [\text{LRFD Eq. 5.9.5.1-1}]$$

However, considering the steel relaxation loss before transfer  $\Delta f_{pR1}$ , the total prestress loss is calculated using the following expression:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$$

The calculation of prestress loss due to elastic shortening, steel relaxation before and after transfer, creep of concrete and shrinkage of concrete are shown in following sections.

Trial number of strands = 48

A number of iterations based on TxDOT methodology (TxDOT 2001) will be performed to arrive at the optimum number of strands, required concrete strength at release ( $f'_{ci}$ ), and required concrete strength at service ( $f'_c$ ).

### **A.2.7.1**

#### **Iteration 1**

#### **A.2.7.1.1**

#### **Elastic Shortening**

[LRFD Art. 5.9.5.2.3]

The loss in prestress due to elastic shortening in prestressed members is given as

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \quad [\text{LRFD Eq. 5.9.5.2.3a-1}]$$

where:

$E_p$  = Modulus of elasticity of prestressing steel = 28,500 ksi

$E_{ci}$  = Modulus of elasticity of girder concrete at transfer, ksi  
 $= 33,000(w_c)^{1.5} \sqrt{f'_{ci}}$  [LRFD Eq. 5.4.2.4-1]

$w_c$  = Unit weight of concrete (must be between 0.09 and 0.155 kcf for LRFD Eq. 5.4.2.4-1 to be applicable)  
 $= 0.150$  kcf

$f'_{ci}$  = Initial estimate of compressive strength of girder concrete at release = 4 ksi

$$E_{ci} = [33,000(0.150)^{1.5} \sqrt{4}] = 3834.25 \text{ ksi}$$

$f_{cgp}$  = Sum of concrete stresses at the center of gravity of the prestressing steel due to prestressing force at transfer and the self-weight of the member at sections of maximum moment, ksi

$$= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I}$$

$P_i$  = Pretension force after allowing for the initial losses, kips

$A$  = Area of girder cross-section = 788.4 in.<sup>2</sup>

$I$  = Moment of inertia of the non-composite section  
= 260,403 in.<sup>4</sup>

$e_c$  = Eccentricity of the prestressing strands at the midspan  
= 19.67 in.

$M_g$  = Moment due to girder self-weight at midspan, k-ft.  
= 1209.98 k-ft.

LRFD Art. 5.9.5.2.3a states that for pretensioned components of usual design,  $f_{cgp}$  can be calculated on the basis of prestressing steel stress assumed to be  $0.7f_{pu}$  for low-relaxation strands. However, TxDOT methodology is to assume the initial losses as a percentage of the initial prestressing stress before release,  $f_{pi}$ . In both procedures, initial losses assumed has to be checked, and if different from the assumed value, a second iteration should be carried out.

TxDOT methodology is used in this example, and initial loss is assumed to be 8 percent of initial prestress,  $f_{pi}$ .

$P_i$  = Pretension force after allowing for 8 percent initial loss, kips

$$= (\text{number of strands})(\text{area of each strand})[0.92(f_{pi})]$$

$$= 48(0.153)(0.92)(202.5) = 1368.19 \text{ kips}$$

$$f_{cgp} = \frac{1368.19}{788.4} + \frac{1368.19(19.67)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.67)}{260,403}$$

$$= 1.735 + 2.033 - 1.097 = 2.671 \text{ ksi}$$

Prestress loss due to elastic shortening is:

$$\Delta f_{pES} = \left[ \frac{28,500}{3834.25} \right] (2.671) = 19.854 \text{ ksi}$$

[LRFD Art. 5.9.5.4.2]

### **A.2.7.1.2 Concrete Shrinkage**

The loss in prestress due to concrete shrinkage, for pretensioned members is given as:

$$\Delta f_{pSR} = 17 - 0.15 H \quad [\text{LRFD Eq. 5.9.5.4.2-1}]$$

where:

$H$  = Average annual ambient relative humidity = 60 percent

$$\Delta f_{pSR} = [17 - 0.15(60)] = 8.0 \text{ ksi}$$

[LRFD Art. 5.9.5.4.3]

### **A.2.7.1.3 Creep of Concrete**

The loss in prestress due to creep of concrete is given as:

$$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp} \geq 0 \quad [\text{LRFD Eq. 5.9.5.4.3-1}]$$

where:

$\Delta f_{cdp}$  = Change in concrete stress at the center of gravity of the prestressing steel due to permanent loads except the dead load present at the time the prestress force is applied, calculated at the same section as  $f_{cgp}$

$$= \frac{M_S e_c}{I} + \frac{M_{SDL} (y_{bc} - y_{bs})}{I_c}$$

$M_S$  = Moment due to slab weight at the midspan section  
= 1179.03 k-ft.

$M_{SDL}$  = Moment due to superimposed dead load  
=  $M_{barr} + M_{DW}$

$M_{barr}$  = Moment due to barrier weight = 160.64 k-ft.

$M_{DW}$  = Moment due to wearing surface load = 188.64 k-ft.

$$M_{SDL} = 160.64 + 188.64 = 349.28 \text{ k-ft.}$$

$y_{bc}$  = Distance from the centroid of the composite section to the extreme bottom fiber of the precast girder = 41.157 in.

$y_{bs}$  = Distance from center of gravity of the prestressing strands at midspan to the bottom fiber of the girder  
 $= 24.75 - 19.67 = 5.08$  in.

$I$  = Moment of inertia of the non-composite section  
 $= 260,403$  in.<sup>4</sup>

$I_c$  = Moment of inertia of composite section = 694,599.5 in.<sup>4</sup>

$$\begin{aligned}\Delta f_{cdp} &= \frac{1179.03(12 \text{ in./ft.})(19.67)}{260,403} \\ &\quad + \frac{(349.28)(12 \text{ in./ft.})(41.157 - 5.08)}{694,599.5} \\ &= 1.069 + 0.218 = 1.287 \text{ ksi}\end{aligned}$$

Prestress loss due to creep of concrete is:  
 $\Delta f_{pCR} = 12(2.671) - 7(1.287) = 23.05$  ksi

**A.2.7.1.4**  
**Relaxation of**  
**Prestressing Strands**  
**A.2.7.1.4.1**  
**Relaxation at**  
**Transfer**

[LRFD Art. 5.9.5.4.4]

[LRFD Art. 5.9.5.4.4b]

For pretensioned members with low-relaxation prestressing steel, initially stressed in excess of  $0.5f_{pu}$ , the relaxation loss is given as:

$$\Delta f_{pRI} = \frac{\log(24.0t)}{40} \left[ \frac{f_{pj}}{f_{py}} - 0.55 \right] f_{pj} \quad [\text{LRFD Eq. 5.9.5.4.4b-2}]$$

where:

$\Delta f_{pRI}$  = Prestress loss due to relaxation of steel at transfer, ksi

$f_{pu}$  = Ultimate stress in prestressing steel = 270 ksi

$f_{pj}$  = Initial stress in tendon at the end of stressing  
 $= 0.75f_{pu} = 0.75(270) = 202.5$  ksi  $> 0.5f_{pu} = 135$  ksi

$t$  = Time estimated in days from stressing to transfer taken as 1 day [default value for PSTRS14 design program (TxDOT 2004)]

$f_{py}$  = Yield strength of prestressing steel = 243 ksi

Prestress loss due to initial steel relaxation is

$$\Delta f_{pRI} = \frac{\log(24.0)(1)}{40} \left[ \frac{202.5}{243} - 0.55 \right] 202.5 = 1.98 \text{ ksi}$$



**A.2.7.1.4.2**  
**Relaxation after**  
**Transfer**

[LRFD Art. 5.9.5.4.4c]

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is given as:

$$\Delta f_{pR2} = 30\% \text{ of } [20.0 - 0.4 \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})]$$

[LRFD Art. 5.9.5.4.4c-1]

where the variables are the same as defined in Section A.2.7 expressed in ksi units

$$\Delta f_{pR2} = 0.3[20.0 - 0.4(19.854) - 0.2(8.0 + 23.05)] = 1.754 \text{ ksi}$$

The instantaneous loss of prestress is estimated using the following expression:

$$\begin{aligned} \Delta f_{pi} &= \Delta f_{pES} + \Delta f_{pR1} \\ &= 19.854 + 1.980 = 21.834 \text{ ksi} \end{aligned}$$

The percent instantaneous loss is calculated using the following expression:

$$\begin{aligned} \% \Delta f_{pi} &= \frac{100(\Delta f_{pES} + \Delta f_{pR1})}{f_{pj}} \\ &= \frac{100(19.854 + 1.980)}{202.5} = 10.78\% > 8\% \text{ (assumed value of} \\ &\quad \text{initial prestress loss)} \end{aligned}$$

Therefore, another trial is required assuming 10.78 percent initial prestress loss.

The change in initial prestress loss will not affect the prestress losses due to concrete shrinkage ( $\Delta f_{pSR}$ ) and initial steel relaxation ( $\Delta f_{pR1}$ ). Therefore, the next trial will involve updating the losses due to elastic shortening ( $\Delta f_{pES}$ ), creep of concrete ( $\Delta f_{pCR}$ ), and steel relaxation after transfer ( $\Delta f_{pR2}$ ).

Based on the initial prestress loss value of 10.78 percent, the pretension force after allowing for the initial losses is calculated as follows.

$$\begin{aligned} P_i &= (\text{number of strands})(\text{area of each strand})[0.8922(f_{pi})] \\ &= 48(0.153)(0.8922)(202.5) = 1326.84 \text{ kips} \end{aligned}$$

Loss in prestress due to elastic shortening

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$

$$\begin{aligned} f_{cgp} &= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I} \\ &= \frac{1326.84}{788.4} + \frac{1326.84(19.67)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.67)}{260,403} \\ &= 1.683 + 1.971 - 1.097 = 2.557 \text{ ksi} \end{aligned}$$

$$E_{ci} = 3834.25 \text{ ksi}$$

$$E_p = 28,500 \text{ ksi}$$

Prestress loss due to elastic shortening is:

$$\Delta f_{pES} = \left[ \frac{28,500}{3834.25} \right] (2.557) = 19.01 \text{ ksi}$$

The loss in prestress due to creep of concrete is given as:

$$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp} \geq 0$$

The value of  $\Delta f_{cdp}$  depends on the dead load moments, superimposed dead load moments, and the section properties. Thus, this value will not change with the change in initial prestress value and will be the same as calculated in Section A.2.7.1.3.

$$\Delta f_{cdp} = 1.287 \text{ ksi}$$

$$\Delta f_{pCR} = 12(2.557) - 7(1.287) = 21.675 \text{ ksi}$$

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is:

$$\begin{aligned} \Delta f_{pR2} &= 30\% \text{ of } [20.0 - 0.4 \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})] \\ &= 0.3[20.0 - 0.4(19.01) - 0.2(8.0 + 21.675)] = 1.938 \text{ ksi} \end{aligned}$$

The instantaneous loss of prestress is estimated using the following expression:

$$\begin{aligned} \Delta f_{pi} &= \Delta f_{pES} + \Delta f_{pR1} \\ &= 19.01 + 1.980 = 20.99 \text{ ksi} \end{aligned}$$

The percent instantaneous loss is calculated using the following expression:

$$\begin{aligned}\% \Delta f_{pi} &= \frac{100(\Delta f_{pES} + \Delta f_{pRI})}{f_{pj}} \\ &= \frac{100(19.01 + 1.980)}{202.5} = 10.37\% < 10.78\% \text{ (assumed value of initial prestress loss)}\end{aligned}$$

Therefore, another trial is required assuming 10.37 percent initial prestress loss.

Based on the initial prestress loss value of 10.37 percent, the pretension force after allowing for the initial losses is calculated as follows.

$$\begin{aligned}P_i &= (\text{number of strands})(\text{area of each strand})[0.8963(f_{pj})] \\ &= 48(0.153)(0.8963)(202.5) = 1332.94 \text{ kips}\end{aligned}$$

Loss in prestress due to elastic shortening

$$\begin{aligned}\Delta f_{pES} &= \frac{E_p}{E_{ci}} f_{cgp} \\ f_{cgp} &= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I} \\ &= \frac{1332.94}{788.4} + \frac{1332.94(19.67)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.67)}{260,403} \\ &= 1.691 + 1.980 - 1.097 = 2.574 \text{ ksi} \\ E_{ci} &= 3834.25 \text{ ksi} \\ E_p &= 28,500 \text{ ksi}\end{aligned}$$

Prestress loss due to elastic shortening is

$$\Delta f_{pES} = \left[ \frac{28,500}{3834.25} \right] (2.574) = 19.13 \text{ ksi}$$

The loss in prestress due to creep of concrete is given as:

$$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp} \geq 0$$

$$\Delta f_{cdp} = 1.287 \text{ ksi}$$

$$\Delta f_{pCR} = 12(2.574) - 7(1.287) = 21.879 \text{ ksi}$$

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is:

$$\Delta f_{pR2} = 30\% \text{ of } [20.0 - 0.4 \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})]$$

$$= 0.3[20.0 - 0.4(19.13) - 0.2(8.0 + 21.879)] = 1.912 \text{ ksi}$$

The instantaneous loss of prestress is estimated using the following expression:

$$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pRI}$$

$$= 19.13 + 1.98 = 21.11 \text{ ksi}$$

The percent instantaneous loss is calculated using the following expression:

$$\% \Delta f_{pi} = \frac{100(\Delta f_{pES} + \Delta f_{pRI})}{f_{pi}}$$

$$= \frac{100(19.13 + 1.98)}{202.5} = 10.42\% \approx 10.37\% \text{ (assumed value of initial prestress loss)}$$

#### **A.2.7.1.5** **Total Losses at Transfer**

Total prestress loss at transfer

$$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pRI}$$

$$= 19.13 + 1.98 = 21.11 \text{ ksi}$$

Effective initial prestress,  $f_{pi} = 202.5 - 21.11 = 181.39 \text{ ksi}$

$P_i$  = Effective pretension after allowing for the initial prestress loss

$$= (\text{number of strands})(\text{area of each strand})(f_{pi})$$

$$= 48(0.153)(181.39) = 1332.13 \text{ kips}$$

#### **A.2.7.1.6** **Total Losses at Service Loads**

Total final loss in prestress:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pRI} + \Delta f_{pR2}$$

$$\Delta f_{pES} = \text{Prestress loss due to elastic shortening} = 19.13 \text{ ksi}$$

$$\Delta f_{pSR} = \text{Prestress loss due to concrete shrinkage} = 8.0 \text{ ksi}$$

$$\Delta f_{pCR} = \text{Prestress loss due to concrete creep} = 21.879 \text{ ksi}$$

$$\Delta f_{pRI} = \text{Prestress loss due to steel relaxation before transfer}$$

$$= 1.98 \text{ ksi}$$

$$\Delta f_{pR2} = \text{Prestress loss due to steel relaxation after transfer}$$

$$= 1.912 \text{ ksi}$$

$$\Delta f_{pT} = 19.13 + 8.0 + 21.879 + 1.98 + 1.912 = 52.901 \text{ ksi}$$

The percent final loss is calculated using the following expression:

$$\begin{aligned} \% \Delta f_{pT} &= \frac{100(\Delta f_{pT})}{f_{pj}} \\ &= \frac{100(52.901)}{202.5} = 26.12\% \end{aligned}$$

Effective final prestress

$$f_{pe} = f_{pj} - \Delta f_{pT} = 202.5 - 52.901 = 149.60 \text{ ksi}$$

Check prestressing stress limit at service limit state (defined in Section A.2.3):  $f_{pe} \leq 0.8f_{py}$

$$f_{py} = \text{Yield strength of prestressing steel} = 243 \text{ ksi}$$

$$f_{pe} = 149.60 \text{ ksi} < 0.8(243) = 194.4 \text{ ksi} \quad (\text{O.K.})$$

Effective prestressing force after allowing for final prestress loss

$$\begin{aligned} P_{pe} &= (\text{number of strands})(\text{area of each strand})(f_{pe}) \\ &= 48(0.153)(149.60) = 1098.66 \text{ kips} \end{aligned}$$

#### **A.2.7.1.7 Final Stresses at Midspan**

The number of strands is updated based on the final stress at the bottom fiber of the girder at the midspan section.

Final stress at the bottom fiber of the girder at the midspan section due to effective prestress ( $f_{bf}$ ) is calculated as follows:

$$\begin{aligned} f_{bf} &= \frac{P_{pe}}{A} + \frac{P_{pe} e_c}{S_b} \\ &= \frac{1098.66}{788.4} + \frac{1098.66(19.67)}{10,521.33} \\ &= 1.393 + 2.054 = 3.447 \text{ ksi} < f_{pb-reqd.} = 3.700 \text{ ksi} \quad (\text{N.G}) \end{aligned}$$

( $f_{pb-reqd.}$  calculations are presented in Section A.2.6.3.)

Try 50 – 0.5 in. diameter, low-relaxation strands.

Eccentricity of prestressing strands at midspan

$$e_c = 24.75 - \frac{12(2 + 4 + 6) + 10(8) + 4(10)}{50} = 19.47 \text{ in.}$$

Effective pretension after allowing for the final prestress loss

$$P_{pe} = 50(0.153)(149.60) = 1144.44 \text{ kips}$$

Final stress at the bottom fiber of the girder at the midspan section due to effective prestress ( $f_{bf}$ ) is:

$$\begin{aligned} f_{bf} &= \frac{1144.44}{788.4} + \frac{1144.44(19.47)}{10,521.33} \\ &= 1.452 + 2.118 = 3.57 \text{ ksi} < f_{pb-reqd.} = 3.700 \text{ ksi} \quad (\text{N.G}) \end{aligned}$$

Try 52 – 0.5 in. diameter, low-relaxation strands.

Eccentricity of prestressing strands at midspan

$$e_c = 24.75 - \frac{12(2 + 4 + 6) + 10(8) + 6(10)}{52} = 19.29 \text{ in.}$$

Effective pretension after allowing for the final prestress loss

$$P_{pe} = 52(0.153)(149.60) = 1190.22 \text{ kips}$$

Final stress at the bottom fiber of the girder at the midspan section due to effective prestress ( $f_{bf}$ ) is:

$$\begin{aligned} f_{bf} &= \frac{1190.22}{788.4} + \frac{1190.22(19.29)}{10,521.33} \\ &= 1.509 + 2.182 = 3.691 \text{ ksi} < f_{pb-reqd.} = 3.700 \text{ ksi} \quad (\text{N.G}) \end{aligned}$$

Try 54 – 0.5 in. diameter, low-relaxation strands.

Eccentricity of prestressing strands at midspan

$$e_c = 24.75 - \frac{12(2 + 4 + 6) + 10(8) + 8(10)}{54} = 19.12 \text{ in.}$$

Effective pretension after allowing for the final prestress loss

$$P_{pe} = 54(0.153)(149.60) = 1236.0 \text{ kips}$$

Final stress at the bottom fiber of the girder at the midspan section due to effective prestress ( $f_{bf}$ ) is:

$$\begin{aligned} f_{bf} &= \frac{1236.0}{788.4} + \frac{1236.0(19.12)}{10,521.33} \\ &= 1.567 + 2.246 = 3.813 \text{ ksi} > f_{pb-reqd.} = 3.700 \text{ ksi} \quad (\text{O.K.}) \end{aligned}$$

Therefore, use 54 – 0.5 in. diameter, 270 ksi low-relaxation strands.

Concrete stress at the top fiber of the girder due to effective prestress and applied permanent and transient loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + f_t = \frac{1236.0}{788.4} - \frac{1236.0(19.12)}{8902.67} + 3.747$$

$$= 1.567 - 2.654 + 3.747 = 2.66 \text{ ksi}$$

( $f_t$  calculations are shown in Section A.2.6.1.)

#### **A.2.7.1.8** **Initial Stresses at** **Hold-Down Point**

The concrete strength at release,  $f'_{ci}$ , is updated based on the initial stress at the bottom fiber of the girder at the hold-down point.

Prestressing force after allowing for initial prestress loss

$$P_i = (\text{number of strands})(\text{area of strand})(\text{effective initial prestress})$$

$$= 54(0.153)(181.39) = 1498.64 \text{ kips}$$

(Effective initial prestress calculations are presented in Section A.2.7.1.5.)

Initial concrete stress at top fiber of the girder at the hold-down point due to self-weight of the girder and effective initial prestress

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{M_g}{S_t}$$

where:

$$M_g = \text{Moment due to girder self-weight at the hold-down point based on overall girder length of } 109'-8''$$

$$= 0.5wx(L - x)$$

$$w = \text{Self-weight of the girder} = 0.821 \text{ kips/ft.}$$

$$L = \text{Overall girder length} = 109.67 \text{ ft.}$$

$$x = \text{Distance of hold-down point from the end of the girder}$$

$$= HD + (\text{distance from centerline of bearing to the girder end})$$

$$HD = \text{Hold-down point distance from centerline of the bearing}$$

$$= 48.862 \text{ ft. (see Sec. A.2.5.1.3)}$$

$$x = 48.862 + 0.542 = 49.404 \text{ ft.}$$

$$M_g = 0.5(0.821)(49.404)(109.67 - 49.404) = 1222.22 \text{ k-ft.}$$

$$f_{ti} = \frac{1498.64}{788.4} - \frac{1498.64(19.12)}{8902.67} + \frac{1222.22(12 \text{ in./ft.})}{8902.67}$$

$$= 1.901 - 3.218 + 1.647 = 0.330 \text{ ksi}$$

Initial concrete stress at bottom fiber of the girder at the hold-down point due to self-weight of the girder and effective initial prestress

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{M_g}{S_b}$$

$$= \frac{1498.64}{788.4} + \frac{1498.64(19.12)}{10,521.33} - \frac{1222.22(12 \text{ in./ft.})}{10,521.33}$$

$$= 1.901 + 2.723 - 1.394 = 3.230 \text{ ksi}$$

Compression stress limit for pretensioned members at transfer stage is  $0.6 f'_{ci}$  [LRFD Art. 5.9.4.1.1]

$$\text{Therefore, } f'_{ci \text{ -reqd.}} = \frac{3,230}{0.6} = 5383.33 \text{ psi}$$

#### **A.2.7.2 Iteration 2**

A second iteration is carried out to determine the prestress losses and to subsequently estimate the required concrete strength at release and at service using the following parameters determined in the previous iteration.

Number of strands = 54

Concrete strength at release,  $f'_{ci} = 5383.33 \text{ psi}$

#### **A.2.7.2.1 Elastic Shortening**

[LRFD Art. 5.9.5.2.3]

The loss in prestress due to elastic shortening in prestressed members is given as:

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \quad [\text{LRFD Eq. 5.9.5.2.3a-1}]$$

where:

$E_p$  = Modulus of elasticity of prestressing steel = 28,500 ksi

$E_{ci}$  = Modulus of elasticity of girder concrete at transfer, ksi  
 $= 33,000(w_c)^{1.5} \sqrt{f'_{ci}}$  [LRFD Eq. 5.4.2.4-1]

$w_c$  = Unit weight of concrete (must be between 0.09 and 0.155 kcf for LRFD Eq. 5.4.2.4-1 to be applicable)  
 $= 0.150 \text{ kcf}$



$$f'_{ci} = \text{Compressive strength of girder concrete at release} \\ = 5.383 \text{ ksi}$$

$$E_{ci} = [33,000(0.150)^{1.5} \sqrt{5.383}] = 4447.98 \text{ ksi}$$

$$f_{cgp} = \text{Sum of concrete stresses at the center of gravity of the prestressing steel due to prestressing force at transfer and the self-weight of the member at sections of maximum moment, ksi}$$

$$= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I}$$

$$A = \text{Area of girder cross-section} = 788.4 \text{ in.}^2$$

$$I = \text{Moment of inertia of the non-composite section} \\ = 260,403 \text{ in.}^4$$

$$e_c = \text{Eccentricity of the prestressing strands at the midspan} \\ = 19.12 \text{ in.}$$

$$M_g = \text{Moment due to girder self-weight at midspan, k-ft.} \\ = 1209.98 \text{ k-ft.}$$

$$P_i = \text{Pretension force after allowing for the initial losses, kips}$$

As the initial losses are dependent on the elastic shortening and the initial steel relaxation loss, which are yet to be determined, the initial loss value of 10.42 percent obtained in the last trial (iteration 1) is taken as an initial estimate for the initial loss in prestress for this iteration.

$$P_i = (\text{number of strands})(\text{area of strand})[0.8958(f_{pi})] \\ = 54(0.153)(0.8958)(202.5) = 1498.72 \text{ kips}$$

$$f_{cgp} = \frac{1498.72}{788.4} + \frac{1498.72(19.12)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.12)}{260,403} \\ = 1.901 + 2.104 - 1.066 = 2.939 \text{ ksi}$$

The prestress loss due to elastic shortening is

$$\Delta f_{pES} = \left[ \frac{28,500}{4447.98} \right] (2.939) = 18.83 \text{ ksi}$$

### A.2.7.2.2 Concrete Shrinkage

[LRFD Art. 5.9.5.4.2]

The loss in prestress due to concrete shrinkage ( $\Delta f_{pSR}$ ) depends on the relative humidity only. The change in compressive strength of girder concrete at release ( $f'_{ci}$ ) and number of strands does not effect the prestress loss due to concrete shrinkage. It will remain the same as calculated in Section A.2.7.1.2.

$$\Delta f_{pSR} = 8.0 \text{ ksi}$$

### A.2.7.2.3 Creep of Concrete

[LRFD Art. 5.9.5.4.3]

The loss in prestress due to creep of concrete is given as:

$$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp} \geq 0 \quad [\text{LRFD Eq. 5.9.5.4.3-1}]$$

where:

$\Delta f_{cdp}$  = Change in concrete stress at the center of gravity of the prestressing steel due to permanent loads except the dead load present at the time the prestress force is applied and calculated at the same section as  $f_{cgp}$ .

$$= \frac{M_S e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

$M_S$  = Moment due to slab weight at midspan section  
= 1179.03 k-ft.

$M_{SDL}$  = Moment due to superimposed dead load  
=  $M_{barr} + M_{DW}$

$M_{barr}$  = Moment due to barrier weight = 160.64 k-ft.

$M_{DW}$  = Moment due to wearing surface load = 188.64 k-ft.

$M_{SDL} = 160.64 + 188.64 = 349.28 \text{ k-ft.}$

$y_{bc}$  = Distance from the centroid of the composite section to extreme bottom fiber of the precast girder = 41.157 in.

$y_{bs}$  = Distance from center of gravity of the prestressing strands at midspan to the bottom fiber of the girder  
=  $24.75 - 19.12 = 5.63 \text{ in.}$

$I$  = Moment of inertia of the non-composite section  
= 260,403 in.<sup>4</sup>

$I_c$  = Moment of inertia of composite section = 694,599.5 in.<sup>4</sup>

$$\begin{aligned}\Delta f_{cdp} &= \frac{1179.03(12 \text{ in./ft.})(19.12)}{260,403} \\ &\quad + \frac{(349.28)(12 \text{ in./ft.})(41.157 - 5.63)}{694,599.5} \\ &= 1.039 + 0.214 = 1.253 \text{ ksi}\end{aligned}$$

Prestress loss due to creep of concrete is

$$\Delta f_{pCR} = 12(2.939) - 7(1.253) = 26.50 \text{ ksi}$$

**A.2.7.2.4**  
**Relaxation of**  
**Prestressing Strands**  
**A.2.7.2.4.1**  
**Relaxation at**  
**Transfer**

[LRFD Art. 5.9.5.4.4]

[LRFD Art. 5.9.5.4.4b]

The loss in prestress due to relaxation of steel at transfer ( $\Delta f_{pR1}$ ) depends on the time from stressing to transfer of prestress ( $t$ ), the initial stress in tendon at the end of stressing ( $f_{pi}$ ), and the yield strength of prestressing steel ( $f_{py}$ ). The change in compressive strength of girder concrete at release ( $f'_{ci}$ ) and number of strands does not affect the prestress loss due to relaxation of steel before transfer. It will remain the same as calculated in Section A.2.7.1.4.1.

$$\Delta f_{pR1} = 1.98 \text{ ksi}$$

**A.2.7.2.4.2**  
**Relaxation after**  
**Transfer**

[LRFD Art. 5.9.5.4.4c]

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is given as:

$$\Delta f_{pR2} = 30\% \text{ of } [20.0 - 0.4 \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})]$$

[LRFD Art. 5.9.5.4.4c-1]

where the variables are same as defined in Section A.2.7 expressed in ksi units

$$\Delta f_{pR2} = 0.3[20.0 - 0.4(18.83) - 0.2(8.0 + 26.50)] = 1.670 \text{ ksi}$$

The instantaneous loss of prestress is estimated using the following expression:

$$\begin{aligned}\Delta f_{pi} &= \Delta f_{pES} + \Delta f_{pR1} \\ &= 18.83 + 1.980 = 20.81 \text{ ksi}\end{aligned}$$

The percent instantaneous loss is calculated using the following expression:

$$\begin{aligned}\% \Delta f_{pi} &= \frac{100(\Delta f_{pES} + \Delta f_{pRI})}{f_{pj}} \\ &= \frac{100(18.83 + 1.98)}{202.5} = 10.28\% < 10.42\% \text{ (assumed value of} \\ &\quad \text{initial prestress loss)}\end{aligned}$$

Therefore, another trial is required assuming 10.28 percent initial prestress loss.

The change in initial prestress loss will not affect the prestress losses due to concrete shrinkage ( $\Delta f_{pSR}$ ) and initial steel relaxation ( $\Delta f_{pRI}$ ). Therefore, the new trials will involve updating the losses due to elastic shortening ( $\Delta f_{pES}$ ), creep of concrete ( $\Delta f_{pCR}$ ), and steel relaxation after transfer ( $\Delta f_{pR2}$ ).

Based on the initial prestress loss value of 10.28 percent, the pretension force after allowing for the initial losses is calculated as follows.

$$\begin{aligned}P_i &= (\text{number of strands})(\text{area of each strand})[0.8972(f_{pi})] \\ &= 54(0.153)(0.8972)(202.5) = 1501.06 \text{ kips}\end{aligned}$$

Loss in prestress due to elastic shortening

$$\begin{aligned}\Delta f_{pES} &= \frac{E_p}{E_{ci}} f_{cgp} \\ f_{cgp} &= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I} \\ &= \frac{1501.06}{788.4} + \frac{1501.06(19.12)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.12)}{260,403} \\ &= 1.904 + 2.107 - 1.066 = 2.945 \text{ ksi} \\ E_{ci} &= 4447.98 \text{ ksi} \\ E_p &= 28,500 \text{ ksi}\end{aligned}$$

Prestress loss due to elastic shortening is

$$\Delta f_{pES} = \left[ \frac{28,500}{4447.98} \right] (2.945) = 18.87 \text{ ksi}$$

The loss in prestress due to creep of concrete is given as:

$$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp} \geq 0$$

The value of  $\Delta f_{cdp}$  depends on the dead load moments, superimposed dead load moments, and section properties. Thus, this value will not change with the change in initial prestress value and will be the same as calculated in Section A.2.7.2.3.

$$\Delta f_{cdp} = 1.253 \text{ ksi}$$

$$\Delta f_{pCR} = 12(2.945) - 7(1.253) = 26.57 \text{ ksi}$$

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is:

$$\begin{aligned} \Delta f_{pR2} &= 30\% \text{ of } [20.0 - 0.4 \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})] \\ &= 0.3[20.0 - 0.4(18.87) - 0.2(8.0 + 26.57)] = 1.661 \text{ ksi} \end{aligned}$$

The instantaneous loss of prestress is estimated using the following expression:

$$\begin{aligned} \Delta f_{pi} &= \Delta f_{pES} + \Delta f_{pR1} \\ &= 18.87 + 1.98 = 20.85 \text{ ksi} \end{aligned}$$

The percent instantaneous loss is calculated using the following expression:

$$\begin{aligned} \% \Delta f_{pi} &= \frac{100(\Delta f_{pES} + \Delta f_{pR1})}{f_{pi}} \\ &= \frac{100(18.87 + 1.98)}{202.5} = 10.30\% \approx 10.28\% \text{ (assumed value of initial prestress loss)} \end{aligned}$$

#### **A.2.7.2.5** **Total Losses at** **Transfer**

Total prestress loss at transfer

$$\begin{aligned} \Delta f_{pi} &= \Delta f_{pES} + \Delta f_{pR1} \\ &= 18.87 + 1.98 = 20.85 \text{ ksi} \end{aligned}$$

$$\text{Effective initial prestress, } f_{pi} = 202.5 - 20.85 = 181.65 \text{ ksi}$$

$P_i$  = Effective pretension after allowing for the initial prestress loss

$$\begin{aligned} &= (\text{number of strands})(\text{area of each strand})(f_{pi}) \\ &= 54(0.153)(181.65) = 1500.79 \text{ kips} \end{aligned}$$

**A.2.7.2.6**  
**Total Losses at**  
**Service Loads**

Total final loss in prestress

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$$

$$\Delta f_{pES} = \text{Prestress loss due to elastic shortening} = 18.87 \text{ ksi}$$

$$\Delta f_{pSR} = \text{Prestress loss due to concrete shrinkage} = 8.0 \text{ ksi}$$

$$\Delta f_{pCR} = \text{Prestress loss due to concrete creep} = 26.57 \text{ ksi}$$

$$\Delta f_{pR1} = \text{Prestress loss due to steel relaxation before transfer} \\ = 1.98 \text{ ksi}$$

$$\Delta f_{pR2} = \text{Prestress loss due to steel relaxation after transfer} \\ = 1.661 \text{ ksi}$$

$$\Delta f_{pT} = 18.87 + 8.0 + 26.57 + 1.98 + 1.661 = 57.08 \text{ ksi}$$

The percent final loss is calculated using the following expression:

$$\begin{aligned} \% \Delta f_{pT} &= \frac{100(\Delta f_{pT})}{f_{pj}} \\ &= \frac{100(57.08)}{202.5} = 28.19\% \end{aligned}$$

Effective final prestress

$$f_{pe} = f_{pj} - \Delta f_{pT} = 202.5 - 57.08 = 145.42 \text{ ksi}$$

Check prestressing stress limit at service limit state (defined in Section A.2.3):  $f_{pe} \leq 0.8f_{py}$

$$f_{py} = \text{Yield strength of prestressing steel} = 243 \text{ ksi}$$

$$f_{pe} = 145.42 \text{ ksi} < 0.8(243) = 194.4 \text{ ksi} \quad (\text{O.K.})$$

Effective prestressing force after allowing for final prestress loss

$$\begin{aligned} P_{pe} &= (\text{number of strands})(\text{area of each strand})(f_{pe}) \\ &= 54(0.153)(145.42) = 1201.46 \text{ kips} \end{aligned}$$

#### A.2.7.2.7 Final Stresses at Midspan

The required concrete strength at service ( $f'_c$  -*reqd.*) is updated based on the final stresses at the top and bottom fibers of the girder at the midspan section shown as follows.

Concrete stresses at the top fiber of the girder at the midspan section due to transient loads, permanent loads, and effective final prestress will be investigated for the following three cases using the Service I limit state shown as follows.

- 1) Concrete stress at the top fiber of the girder at the midspan section due to effective final prestress + permanent loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW}}{S_{tg}}$$

where:

$f_{tf}$  = Concrete stress at the top fiber of the girder, ksi

$M_{DCN}$  = Moment due to non-composite dead loads, k-ft.  
=  $M_g + M_s$

$M_g$  = Moment due to girder self-weight = 1209.98 k-ft.

$M_s$  = Moment due to slab weight = 1179.03 k-ft.

$M_{DCN} = 1209.98 + 1179.03 = 2389.01$  k-ft.

$M_{DCC}$  = Moment due to composite dead loads except wearing surface load, k-ft.  
=  $M_{barr}$

$M_{barr}$  = Moment due to barrier weight = 160.64 k-ft.

$M_{DCC} = 160.64$  k-ft.

$M_{DW}$  = Moment due to wearing surface load = 188.64 k-ft.

$S_t$  = Section modulus referenced to the extreme top fiber of the non-composite precast girder = 8902.67 in.<sup>3</sup>

$S_{tg}$  = Section modulus of composite section referenced to the top fiber of the precast girder = 54,083.9 in.<sup>3</sup>

$$\begin{aligned}
 f_{tf} &= \frac{1201.46}{788.4} - \frac{1201.46(19.12)}{8902.67} + \frac{(2389.01)(12 \text{ in./ft.})}{8902.67} \\
 &\quad + \frac{(160.64 + 188.64)(12 \text{ in./ft.})}{54,083.9} \\
 &= 1.524 - 2.580 + 3.220 + 0.077 = 2.241 \text{ ksi}
 \end{aligned}$$

Compressive stress limit for this service load combination given in LRFD Table 5.9.4.2.1-1 is  $0.45 f'_c$ .

$$f'_{c \text{ -reqd.}} = \frac{2241}{0.45} = 4980.0 \text{ psi (controls)}$$

- 2) Concrete stress at the top fiber of the girder at the midspan section due to live load + 0.5×(effective final prestress + permanent loads)

$$f_{tf} = \frac{(M_{LT} + M_{LL})}{S_{tg}} + 0.5 \left( \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW}}{S_{tg}} \right)$$

where:

$M_{LT}$  = Distributed moment due to HS 20-44 truck load, including dynamic load allowance = 1423.00 k-ft.

$M_{LL}$  = Distributed moment due to lane load = 602.72 k-ft.

$$\begin{aligned}
 f_{tf} &= \frac{(1423 + 602.72)(12 \text{ in./ft.})}{54,083.9} + 0.5 \left\{ \frac{1201.46}{788.4} - \frac{1201.46(19.12)}{8902.67} \right. \\
 &\quad \left. + \frac{(2389.01)(12 \text{ in./ft.})}{8902.67} + \frac{(160.64 + 188.64)(12 \text{ in./ft.})}{54,083.9} \right\} \\
 &= 0.449 + 0.5(1.524 - 2.580 + 3.220 + 0.077) = 1.570 \text{ ksi}
 \end{aligned}$$

Compressive stress limit for this service load combination given in LRFD Table 5.9.4.2.1-1 is  $0.40 f'_c$ .

$$f'_{c \text{ -reqd.}} = \frac{1570}{0.40} = 3925 \text{ psi}$$

- 3) Concrete stress at the top fiber of the girder at the midspan section due to effective prestress + permanent loads + transient loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW} + M_{LT} + M_{LL}}{S_{tg}}$$



$$\begin{aligned}
 f_{tf} &= \frac{1201.46}{788.4} - \frac{1201.46(19.12)}{8902.67} + \frac{(2389.01)(12 \text{ in./ft.})}{8902.67} \\
 &\quad + \frac{(160.64 + 188.64)(12 \text{ in./ft.})}{54,083.9} + \frac{(1423.00 + 602.72)(12 \text{ in./ft.})}{54,083.9} \\
 &= 1.524 - 2.580 + 3.220 + 0.077 + 0.449 = 2.690 \text{ ksi}
 \end{aligned}$$

Compressive stress limit for this service load combination given in LRFD Table 5.9.4.2.1-1 is  $0.60\phi_w f'_c$ .

where  $\phi_w$  is the reduction factor, applicable to thin-walled hollow rectangular compression members where the web or flange slenderness ratios are greater than 15.

[LRFD Art. 5.9.4.2.1]

The reduction factor  $\phi_w$  is not defined for I-shaped girder cross-sections and is taken as 1.0 in this design.

$$f'_c\text{-reqd.} = \frac{2690}{0.60(1.0)} = 4483.33 \text{ psi}$$

Concrete stresses at the bottom fiber of the girder at the midspan section due to transient loads, permanent loads, and effective final prestress is investigated using Service III limit state as follows.

$$\begin{aligned}
 f_{bf} &= \frac{P_{pe}}{A} + \frac{P_{pe} e_c}{S_b} - f_b \text{ (} f_b \text{ calculations are presented in Sec. A.2.6.1)} \\
 &= \frac{1201.46}{788.4} + \frac{1201.46(19.12)}{10,521.33} - 4.125 \\
 &= 1.524 + 2.183 - 4.125 = -0.418 \text{ ksi}
 \end{aligned}$$

Tensile stress limit in fully prestressed concrete members with bonded prestressing tendons, subjected to not worse than moderate corrosion conditions (assumed in this design example) at service limit state after losses is given by LRFD Table 5.9.4.2.2-1 as  $0.19\sqrt{f'_c}$ .

$$f'_c\text{-reqd.} = 1000 \left( \frac{0.418}{0.19} \right)^2 = 4840.0 \text{ psi}$$

The concrete strength at service is updated based on the final stresses at the midspan section under different loading combinations, as shown above. The governing required concrete strength at service is 4980 psi.

**A.2.7.2.8**  
**Initial Stresses at**  
**Hold-Down Point**

Prestressing force after allowing for initial prestress loss

$$P_i = (\text{number of strands})(\text{area of strand})(\text{effective initial prestress})$$

$$= 54(0.153)(181.65) = 1500.79 \text{ kips}$$

(Effective initial prestress calculations are presented in Section A.2.7.2.5.)

Initial concrete stress at top fiber of the girder at hold-down point due to self-weight of girder and effective initial prestress

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{M_g}{S_t}$$

where:

$$M_g = \text{Moment due to girder self-weight at hold-down point}$$

$$\text{based on overall girder length of } 109'-8''$$

$$= 1222.22 \text{ k-ft. (see Section A.2.7.1.8)}$$

$$f_{ti} = \frac{1500.79}{788.4} - \frac{1500.79(19.12)}{8902.67} + \frac{1222.22(12 \text{ in./ft.})}{8902.67}$$

$$= 1.904 - 3.223 + 1.647 = 0.328 \text{ ksi}$$

Initial concrete stress at bottom fiber of the girder at hold-down point due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1500.79}{788.4} + \frac{1500.79(19.12)}{10,521.33} - \frac{1222.22(12 \text{ in./ft.})}{10,521.33}$$

$$= 1.904 + 2.727 - 1.394 = 3.237 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer stage is  $0.60 f'_{ci}$  [LRFD Art.5.9.4.1.1]

$$f'_{ci \text{ -reqd.}} = \frac{3237}{0.60} = 5395 \text{ psi}$$

**A.2.7.2.9**  
**Initial Stresses at**  
**Girder End**

The initial tensile stress at the top fiber and compressive stress at the bottom fiber of the girder at the girder end section are minimized by harping the web strands at the girder end. Following TxDOT methodology (TxDOT 2001), the web strands are incrementally raised as a unit by 2 inches in each trial. The iterations are repeated until the top and bottom fiber stresses satisfy the allowable stress limits, or the centroid of the topmost row of harped strands is at a distance of 2 inches from the top fiber of the girder, in which case, the concrete strength at release is updated based on the governing stress. The position of the harped web strands, eccentricity of strands at the girder end, top and bottom fiber stresses at the girder end, and the corresponding required concrete strengths are summarized in Table A.2.7.1.

*Table A.2.7.1. Summary of Top and Bottom Stresses at Girder End for Different Harped Strand Positions and Corresponding Required Concrete Strengths.*

| Distance of the centroid of topmost row of harped web strands from |                 | Eccentricity of prestressing strands at girder end (in.) | Top fiber stress (ksi) | Required concrete strength (ksi) | Bottom fiber stress (ksi) | Required concrete strength (ksi) |
|--|-----------------|--|------------------------|----------------------------------|---------------------------|----------------------------------|
| Bottom Fiber (in.)   | Top Fiber (in.) |  |                        |                                  |                           |                                  |
| 10 (no harping)  | 44              | 19.12  | -1.320                 | 30.232                           | 4.631                     | 7.718                            |
| 12   | 42              | 18.75  | -1.257                 | 27.439                           | 4.578                     | 7.630                            |
| 14   | 40              | 18.38  | -1.195                 | 24.781                           | 4.525                     | 7.542                            |
| 16   | 38              | 18.01  | -1.132                 | 22.259                           | 4.472                     | 7.454                            |
| 18   | 36              | 17.64  | -1.070                 | 19.872                           | 4.420                     | 7.366                            |
| 20   | 34              | 17.27  | -1.007                 | 17.620                           | 4.367                     | 7.278                            |
| 22   | 32              | 16.90  | -0.945                 | 15.504                           | 4.314                     | 7.190                            |
| 24   | 30              | 16.53  | -0.883                 | 13.523                           | 4.261                     | 7.102                            |
| 26   | 28              | 16.16  | -0.820                 | 11.677                           | 4.208                     | 7.014                            |
| 28   | 26              | 15.79  | -0.758                 | 9.967                            | 4.155                     | 6.926                            |
| 30   | 24              | 15.42  | -0.695                 | 8.392                            | 4.103                     | 6.838                            |
| 32   | 22              | 15.05  | -0.633                 | 6.952                            | 4.050                     | 6.750                            |
| 34   | 20              | 14.68  | -0.570                 | 5.648                            | 3.997                     | 6.662                            |
| 36   | 18              | 14.31  | -0.508                 | 4.479                            | 3.944                     | 6.574                            |
| 38   | 16              | 13.93  | -0.446                 | 3.446                            | 3.891                     | 6.485                            |
| 40   | 14              | 13.56  | -0.383                 | 2.548                            | 3.838                     | 6.397                            |
| 42   | 12              | 13.19  | -0.321                 | 1.785                            | 3.786                     | 6.309                            |
| 44   | 10              | 12.82  | -0.258                 | 1.157                            | 3.733                     | 6.221                            |
| 46   | 8               | 12.45  | -0.196                 | 0.665                            | 3.680                     | 6.133                            |
| 48   | 6               | 12.08  | -0.133                 | 0.309                            | 3.627                     | 6.045                            |
| 50   | 4               | 11.71  | -0.071                 | 0.087                            | 3.574                     | 5.957                            |
| 52   | 2               | 11.34  | -0.008                 | 0.001                            | 3.521                     | 5.869                            |

The required concrete strengths used in Table A.2.7.1 are based on the allowable stress limits at transfer stage specified in LRFD Art. 5.9.4.1, presented as follows.

$$\text{Allowable compressive stress limit} = 0.60 f'_{ci}$$

For fully prestressed members, in areas with bonded reinforcement sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of  $0.5f_y$  ( $f_y$  is the yield strength of nonprestressed reinforcement), not to exceed 30 ksi, the allowable tension at transfer stage is given as  $0.24\sqrt{f'_{ci}}$

From Table A.2.7.1, it is evident that the web strands are needed to be harped to the topmost position possible to control the bottom fiber stress at the girder end.

Detailed calculations for the case when 10 web strands (5 rows) are harped to the topmost location (centroid of the topmost row of harped strands is at a distance of 2 inches from the top fiber of the girder) is presented as follows.

Eccentricity of prestressing strands at the girder end (see Figure A.2.7.2)

$$\begin{aligned} e_e &= 24.75 - \frac{10(2+4+6) + 8(8) + 6(10) + 2(52+50+48+46+44)}{54} \\ &= 11.34 \text{ in.} \end{aligned}$$

Concrete stress at the top fiber of the girder at the girder end at transfer stage:

$$\begin{aligned} f_{ti} &= \frac{P_i}{A} - \frac{P_i e_e}{S_t} \\ &= \frac{1500.79}{788.4} - \frac{1500.79(11.34)}{8902.67} = 1.904 - 1.912 = -0.008 \text{ ksi} \end{aligned}$$

Tensile stress limit for fully prestressed concrete members with bonded reinforcement is  $0.24\sqrt{f'_{ci}}$  [LRFD Art. 5.9.4.1]

$$f'_{ci \text{ -reqd.}} = 1000 \left( \frac{0.008}{0.24} \right)^2 = 1.11 \text{ psi}$$

Concrete stress at the bottom fiber of the girder at the girder end at transfer stage:

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_e}{S_b}$$

$$= \frac{1500.79}{788.4} + \frac{1500.79 (11.34)}{10,521.33} = 1.904 + 1.618 = 3.522 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer stage is  $0.60 f'_{ci}$  [LRFD Art. 5.9.4.1]

$$f'_{ci \text{ -reqd.}} = \frac{3522}{0.60} = 5870 \text{ psi} \quad (\text{controls})$$

The required concrete strengths are updated based on the above results as follows.

Concrete strength at release,  $f'_{ci} = 5870 \text{ psi}$

Concrete strength at service,  $f'_c$  is greater of 4980 psi and  $f'_{ci}$   
 $f'_c = 5870 \text{ psi}$

### **A.2.7.3 Iteration 3**

A third iteration is carried out to refine the prestress losses based on the updated concrete strengths. Based on the updated prestress losses, the concrete strength at release and at service will be further refined.

Number of strands = 54

Concrete strength at release,  $f'_{ci} = 5870 \text{ psi}$

### **A.2.7.3.1 Elastic Shortening**

[LRFD Art. 5.9.5.2.3]

The loss in prestress due to elastic shortening in prestressed concrete members is given as

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \quad [\text{LRFD Eq. 5.9.5.2.3a-1}]$$

where:

$E_p$  = Modulus of elasticity of prestressing steel = 28,500 ksi

$E_{ci}$  = Modulus of elasticity of girder concrete at transfer, ksi  
 $= 33,000(w_c)^{1.5} \sqrt{f'_{ci}}$  [LRFD Eq. 5.4.2.4-1]

$w_c$  = Unit weight of concrete (must be between 0.09 and 0.155 kcf for LRFD Eq. 5.4.2.4-1 to be applicable)  
 $= 0.150 \text{ kcf}$

$$f'_{ci} = \text{Compressive strength of girder concrete at release} \\ = 5.870 \text{ ksi}$$

$$E_{ci} = [33,000(0.150)^{1.5} \sqrt{5.870}] = 4644.83 \text{ ksi}$$

$$f_{cgp} = \text{Sum of concrete stresses at the center of gravity of the prestressing steel due to prestressing force at transfer and the self-weight of the member at sections of maximum moment, ksi}$$

$$= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I}$$

$$A = \text{Area of girder cross-section} = 788.4 \text{ in.}^2$$

$$I = \text{Moment of inertia of the non-composite section} \\ = 260,403 \text{ in.}^4$$

$$e_c = \text{Eccentricity of the prestressing strands at the midspan} \\ = 19.12 \text{ in.}$$

$$M_g = \text{Moment due to girder self-weight at midspan, k-ft.} \\ = 1209.98 \text{ k-ft.}$$

$$P_i = \text{Pretension force after allowing for the initial losses, kips}$$

As the initial losses are dependent on the elastic shortening and the initial steel relaxation loss, which are yet to be determined, the initial loss value of 10.30 percent obtained in the last trial (iteration 2) is taken as an initial estimate for initial loss in prestress for this iteration.

$$P_i = (\text{number of strands})(\text{area of strand})[0.897(f_{pi})] \\ = 54(0.153)(0.897)(202.5) = 1500.73 \text{ kips}$$

$$f_{cgp} = \frac{1500.73}{788.4} + \frac{1500.73 (19.12)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.12)}{260,403} \\ = 1.904 + 2.107 - 1.066 = 2.945 \text{ ksi}$$

The prestress loss due to elastic shortening is

$$\Delta f_{pES} = \left[ \frac{28,500}{4644.83} \right] (2.945) = 18.07 \text{ ksi}$$

### A.2.7.3.2 Concrete Shrinkage

[LRFD Art. 5.9.5.4.2]

The loss in prestress due to concrete shrinkage ( $\Delta f_{pSR}$ ) depends on the relative humidity only. The change in compressive strength of girder concrete at release ( $f'_{ci}$ ) does not affect the prestress loss due to concrete shrinkage. It will remain the same as calculated in Section A.2.7.1.2.

$$\Delta f_{pSR} = 8.0 \text{ ksi}$$

### A.2.7.3.3 Creep of Concrete

[LRFD Art. 5.9.5.4.3]

The loss in prestress due to creep of concrete is given as:

$$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp} \geq 0 \quad [\text{LRFD Eq. 5.9.5.4.3-1}]$$

where:

$\Delta f_{cdp}$  = Change in concrete stress at the center of gravity of the prestressing steel due to permanent loads except the dead load present at the time the prestress force is applied calculated at the same section as  $f_{cgp}$ .

$$= \frac{M_S e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

$M_S$  = Moment due to the slab weight at midspan section  
= 1179.03 k-ft.

$M_{SDL}$  = Moment due to superimposed dead load  
=  $M_{barr} + M_{DW}$

$M_{barr}$  = Moment due to barrier weight = 160.64 k-ft.

$M_{DW}$  = Moment due to wearing surface load = 188.64 k-ft.

$M_{SDL} = 160.64 + 188.64 = 349.28 \text{ k-ft.}$

$y_{bc}$  = Distance from the centroid of the composite section to the extreme bottom fiber of the precast girder = 41.157 in.

$y_{bs}$  = Distance from centroid of the prestressing strands at midspan to the bottom fiber of the girder  
=  $24.75 - 19.12 = 5.63 \text{ in.}$

$I$  = Moment of inertia of the non-composite section  
= 260,403 in.<sup>4</sup>

$I_c$  = Moment of inertia of composite section = 694,599.5 in.<sup>4</sup>

$$\begin{aligned}\Delta f_{cdp} &= \frac{1179.03(12 \text{ in./ft.})(19.12)}{260,403} \\ &\quad + \frac{(349.28)(12 \text{ in./ft.})(41.157 - 5.63)}{694,599.5} \\ &= 1.039 + 0.214 = 1.253 \text{ ksi}\end{aligned}$$

Prestress loss due to creep of concrete is

$$\Delta f_{pCR} = 12(2.945) - 7(1.253) = 26.57 \text{ ksi}$$

**A.2.7.3.4**  
**Relaxation of**  
**Prestressing Strands**  
**A.2.7.3.4.1**  
**Relaxation at**  
**Transfer**

[LRFD Art. 5.9.5.4.4]

[LRFD Art. 5.9.5.4.4b]

The loss in prestress due to relaxation of steel at transfer ( $\Delta f_{pR1}$ ) depends on the time from stressing to transfer of prestress ( $t$ ), the initial stress in tendon at the end of stressing ( $f_{pi}$ ), and the yield strength of prestressing steel ( $f_{py}$ ). The change in compressive strength of girder concrete at release ( $f'_{ci}$ ) and number of strands does not affect the prestress loss due to relaxation of steel before transfer. It will remain the same as calculated in Section A.2.7.1.4.1.

$$\Delta f_{pR1} = 1.98 \text{ ksi}$$

**A.2.7.3.4.2**  
**Relaxation after**  
**Transfer**

[LRFD Art. 5.9.5.4.4c]

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is given as:

$$\Delta f_{pR2} = 30\% \text{ of } [20.0 - 0.4 \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})]$$

[LRFD Art. 5.9.5.4.4c-1]

where the variables are same as defined in Section A.2.7 expressed in ksi units

$$\Delta f_{pR2} = 0.3[20.0 - 0.4(18.07) - 0.2(8.0 + 26.57)] = 1.757 \text{ ksi}$$

The instantaneous loss of prestress is estimated using the following expression:

$$\begin{aligned}\Delta f_{pi} &= \Delta f_{pES} + \Delta f_{pR1} \\ &= 18.07 + 1.980 = 20.05 \text{ ksi}\end{aligned}$$



The percent instantaneous loss is calculated using the following expression:

$$\begin{aligned}\% \Delta f_{pi} &= \frac{100(\Delta f_{pES} + \Delta f_{pRI})}{f_{pj}} \\ &= \frac{100(18.07 + 1.98)}{202.5} = 9.90\% < 10.30\% \text{ (assumed value of} \\ &\quad \text{initial prestress loss)}\end{aligned}$$

Therefore, another trial is required assuming 9.90 percent initial prestress loss.

The change in initial prestress loss will not affect the prestress losses due to concrete shrinkage ( $\Delta f_{pSR}$ ) and initial steel relaxation ( $\Delta f_{pRI}$ ). Therefore, the new trials will involve updating the losses due to elastic shortening ( $\Delta f_{pES}$ ), creep of concrete ( $\Delta f_{pCR}$ ), and steel relaxation after transfer ( $\Delta f_{pR2}$ ).

Based on the initial prestress loss value of 9.90 percent, the pretension force after allowing for the initial losses is calculated as follows.

$$\begin{aligned}P_i &= (\text{number of strands})(\text{area of each strand})[0.901(f_{pi})] \\ &= 54(0.153)(0.901)(202.5) = 1507.42 \text{ kips}\end{aligned}$$

Loss in prestress due to elastic shortening

$$\begin{aligned}\Delta f_{pES} &= \frac{E_p}{E_{ci}} f_{cgp} \\ f_{cgp} &= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g)e_c}{I} \\ &= \frac{1507.42}{788.4} + \frac{1507.42(19.12)^2}{260,403} - \frac{1209.98(12 \text{ in./ft.})(19.12)}{260,403} \\ &= 1.912 + 2.116 - 1.066 = 2.962 \text{ ksi} \\ E_{ci} &= 4644.83 \text{ ksi} \\ E_p &= 28,500 \text{ ksi}\end{aligned}$$

Prestress loss due to elastic shortening is

$$\Delta f_{pES} = \left[ \frac{28,500}{4644.83} \right] (2.962) = 18.17 \text{ ksi}$$

The loss in prestress due to creep of concrete is given as:

$$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp} \geq 0$$

The value of  $\Delta f_{cdp}$  depends on the dead load moments, superimposed dead load moments, and section properties. Thus, this value will not change with the change in initial prestress value and will be same as calculated in Section A.2.7.2.3.

$$\Delta f_{cdp} = 1.253 \text{ ksi}$$

$$\Delta f_{pCR} = 12(2.962) - 7(1.253) = 26.773 \text{ ksi}$$

For pretensioned members with low-relaxation strands, the prestress loss due to relaxation of steel after transfer is:

$$\begin{aligned} \Delta f_{pR2} &= 30\% \text{ of } [20.0 - 0.4 \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})] \\ &= 0.3[20.0 - 0.4(18.17) - 0.2(8.0 + 26.773)] = 1.733 \text{ ksi} \end{aligned}$$

The instantaneous loss of prestress is estimated using the following expression:

$$\begin{aligned} \Delta f_{pi} &= \Delta f_{pES} + \Delta f_{pRI} \\ &= 18.17 + 1.98 = 20.15 \text{ ksi} \end{aligned}$$

The percent instantaneous loss is calculated using the following expression:

$$\begin{aligned} \% \Delta f_{pi} &= \frac{100(\Delta f_{pES} + \Delta f_{pRI})}{f_{pi}} \\ &= \frac{100(18.17 + 1.98)}{202.5} = 9.95\% \approx 9.90\% \text{ (assumed value of initial prestress loss)} \end{aligned}$$

#### **A.2.7.3.5 Total Losses at Transfer**

Total prestress loss at transfer

$$\begin{aligned} \Delta f_{pi} &= \Delta f_{pES} + \Delta f_{pRI} \\ &= 18.17 + 1.98 = 20.15 \text{ ksi} \end{aligned}$$

$$\text{Effective initial prestress, } f_{pi} = 202.5 - 20.15 = 182.35 \text{ ksi}$$

$P_i$  = Effective pretension after allowing for the initial prestress loss

$$\begin{aligned} &= (\text{number of strands})(\text{area of each strand})(f_{pi}) \\ &= 54(0.153)(182.35) = 1506.58 \text{ kips} \end{aligned}$$

**A.2.7.3.6**  
**Total Losses at**  
**Service Loads**

Total final loss in prestress

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$$

$$\Delta f_{pES} = \text{Prestress loss due to elastic shortening} = 18.17 \text{ ksi}$$

$$\Delta f_{pSR} = \text{Prestress loss due to concrete shrinkage} = 8.0 \text{ ksi}$$

$$\Delta f_{pCR} = \text{Prestress loss due to concrete creep} = 26.773 \text{ ksi}$$

$$\Delta f_{pR1} = \text{Prestress loss due to steel relaxation before transfer} \\ = 1.98 \text{ ksi}$$

$$\Delta f_{pR2} = \text{Prestress loss due to steel relaxation after transfer} \\ = 1.733 \text{ ksi}$$

$$\Delta f_{pT} = 18.17 + 8.0 + 26.773 + 1.98 + 1.773 = 56.70 \text{ ksi}$$

The percent final loss is calculated using the following expression:

$$\begin{aligned} \% \Delta f_{pT} &= \frac{100(\Delta f_{pT})}{f_{pj}} \\ &= \frac{100(56.70)}{202.5} = 28.0\% \end{aligned}$$

Effective final prestress

$$f_{pe} = f_{pj} - \Delta f_{pT} = 202.5 - 56.70 = 145.80 \text{ ksi}$$

Check prestressing stress limit at service limit state (defined in Section A.2.3):  $f_{pe} \leq 0.8f_{py}$

$$f_{py} = \text{Yield strength of prestressing steel} = 243 \text{ ksi}$$

$$f_{pe} = 145.80 \text{ ksi} < 0.8(243) = 194.4 \text{ ksi} \quad (\text{O.K.})$$

Effective prestressing force after allowing for final prestress loss

$$\begin{aligned} P_{pe} &= (\text{number of strands})(\text{area of each strand})(f_{pe}) \\ &= 54(0.153)(145.80) = 1204.60 \text{ kips} \end{aligned}$$

### A.2.7.3.7 Final Stresses at Midspan

The required concrete strength at service ( $f'_c$  -*reqd.*) is updated based on the final stresses at the top and bottom fibers of the girder at the midspan section shown as follows.

Concrete stresses at the top fiber of the girder at the midspan section due to transient loads, permanent loads, and effective final prestress will be investigated for the following three cases using the Service I limit state shown as follows.

- 1) Concrete stress at the top fiber of the girder at the midspan section due to effective final prestress + permanent loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW}}{S_{tg}}$$

where:

$f_{tf}$  = Concrete stress at the top fiber of the girder, ksi

$M_{DCN}$  = Moment due to non-composite dead loads, k-ft.  
=  $M_g + M_s$

$M_g$  = Moment due to girder self-weight = 1209.98 k-ft.

$M_s$  = Moment due to slab weight = 1179.03 k-ft.

$M_{DCN} = 1209.98 + 1179.03 = 2389.01$  k-ft.

$M_{DCC}$  = Moment due to composite dead loads except wearing surface load, k-ft.  
=  $M_{barr}$

$M_{barr}$  = Moment due to barrier weight = 160.64 k-ft.

$M_{DCC} = 160.64$  k-ft.

$M_{DW}$  = Moment due to wearing surface load = 188.64 k-ft.

$S_t$  = Section modulus referenced to the extreme top fiber of the non-composite precast girder = 8902.67 in.<sup>3</sup>

$S_{tg}$  = Section modulus of composite section referenced to the top fiber of the precast girder = 54,083.9 in.<sup>3</sup>

$$\begin{aligned}
 f_{tf} &= \frac{1204.60}{788.4} - \frac{1204.60(19.12)}{8902.67} + \frac{(2389.01)(12 \text{ in./ft.})}{8902.67} \\
 &\quad + \frac{(160.64 + 188.64)(12 \text{ in./ft.})}{54,083.9} \\
 &= 1.528 - 2.587 + 3.220 + 0.077 = 2.238 \text{ ksi}
 \end{aligned}$$

Compressive stress limit for this service load combination given in LRFD Table 5.9.4.2.1-1 is  $0.45 f'_c$ .

$$f'_{c \text{ -reqd.}} = \frac{2238}{0.45} = 4973.33 \text{ psi (controls)}$$

- 2) Concrete stress at the top fiber of the girder at the midspan section due to live load + 0.5×(effective final prestress + permanent loads)

$$f_{tf} = \frac{(M_{LT} + M_{LL})}{S_{tg}} + 0.5 \left( \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW}}{S_{tg}} \right)$$

where:

$M_{LT}$  = Distributed moment due to HS 20-44 truck load including dynamic load allowance = 1423.00 k-ft.

$M_{LL}$  = Distributed moment due to lane load = 602.72 k-ft.

$$\begin{aligned}
 f_{tf} &= \frac{(1423.00 + 602.72)(12 \text{ in./ft.})}{54,083.9} + 0.5 \left\{ \frac{1204.60}{788.4} - \frac{1204.60(19.12)}{8902.67} \right. \\
 &\quad \left. + \frac{(2389.01)(12 \text{ in./ft.})}{8902.67} + \frac{(160.64 + 188.64)(12 \text{ in./ft.})}{54,083.9} \right\} \\
 &= 0.449 + 0.5(1.528 - 2.587 + 3.220 + 0.077) = 1.568 \text{ ksi}
 \end{aligned}$$

Compressive stress limit for this service load combination given in LRFD Table 5.9.4.2.1-1 is  $0.40 f'_c$ .

$$f'_{c \text{ -reqd.}} = \frac{1568}{0.40} = 3920 \text{ psi}$$

- 3) Concrete stress at the top fiber of the girder at the midspan section due to effective prestress + permanent loads + transient loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW} + M_{LT} + M_{LL}}{S_{tg}}$$

$$\begin{aligned}
 f_{tf} &= \frac{1204.60}{788.4} - \frac{1204.60(19.12)}{8902.67} + \frac{(2389.01)(12 \text{ in./ft.})}{8902.67} \\
 &\quad + \frac{(160.64 + 188.64)(12 \text{ in./ft.})}{54,083.9} + \frac{(1423.00 + 602.72)(12 \text{ in./ft.})}{54,083.9} \\
 &= 1.528 - 2.587 + 3.220 + 0.077 + 0.449 = 2.687 \text{ ksi}
 \end{aligned}$$

Compressive stress limit for this service load combination given in LRFD Table 5.9.4.2.1-1 is  $0.60\phi_w f'_c$ .

where  $\phi_w$  is the reduction factor, applicable to thin-walled hollow rectangular compression members where the web or flange slenderness ratios are greater than 15.

[LRFD Art. 5.9.4.2.1]

The reduction factor  $\phi_w$  is not defined for I-shaped girder cross-sections and is taken as 1.0 in this design.

$$f'_{c \text{ -reqd.}} = \frac{2687}{0.60(1.0)} = 4478.33 \text{ psi}$$

Concrete stresses at the bottom fiber of the girder at the midspan section due to transient loads, permanent loads, and effective final prestress will be investigated using Service III limit state as follows.

$$\begin{aligned}
 f_{bf} &= \frac{P_{pe}}{A} + \frac{P_{pe} e_c}{S_b} - f_b \text{ (} f_b \text{ calculations are presented in Sec. A.2.6.1)} \\
 &= \frac{1204.60}{788.4} + \frac{1204.60(19.12)}{10,521.33} - 4.125 \\
 &= 1.528 + 2.189 - 4.125 = -0.408 \text{ ksi}
 \end{aligned}$$

Tensile stress limit in fully prestressed concrete members with bonded prestressing tendons, subjected to not worse than moderate corrosion conditions (assumed in this design example) at service limit state after losses is given by LRFD Table 5.9.4.2.2-1 as  $0.19\sqrt{f'_c}$ .

$$f'_{c \text{ -reqd.}} = 1000 \left( \frac{0.408}{0.19} \right)^2 = 4611 \text{ psi}$$

The concrete strength at service is updated based on the final stresses at the midspan section under different loading combinations as shown above. The governing required concrete strength at service is 4973.33 psi.

**A.2.7.3.8**  
**Initial Stresses at**  
**Hold-Down Point**

Prestressing force after allowing for initial prestress loss

$$P_i = (\text{number of strands})(\text{area of strand})(\text{effective initial prestress})$$

$$= 54(0.153)(182.35) = 1506.58 \text{ kips}$$

(Effective initial prestress calculations are presented in Section A.2.7.3.5.)

Initial concrete stress at top fiber of the girder at hold-down point due to self-weight of girder and effective initial prestress

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{M_g}{S_t}$$

where:

$$M_g = \text{Moment due to girder self-weight at hold-down point}$$

$$\text{based on overall girder length of } 109'-8''$$

$$= 1222.22 \text{ k-ft. (see Section A.2.7.1.8)}$$

$$f_{ti} = \frac{1506.58}{788.4} - \frac{1506.58(19.12)}{8902.67} + \frac{1222.22(12 \text{ in./ft.})}{8902.67}$$

$$= 1.911 - 3.236 + 1.647 = 0.322 \text{ ksi}$$

Initial concrete stress at bottom fiber of the girder at hold-down point due to self-weight of girder and effective initial prestress

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{M_g}{S_b}$$

$$f_{bi} = \frac{1506.58}{788.4} + \frac{1506.58(19.12)}{10,521.33} - \frac{1222.22(12 \text{ in./ft.})}{10,521.33}$$

$$= 1.911 + 2.738 - 1.394 = 3.255 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer stage is  $0.60 f'_{ci}$  [LRFD Art.5.9.4.1.1]

$$f'_{ci \text{ -reqd.}} = \frac{3255}{0.60} = 5425 \text{ psi}$$

**A.2.7.3.9**  
**Initial Stresses at**  
**Girder End**

The eccentricity of the prestressing strands at the girder end when 10 web strands are harped to the topmost location (centroid of the topmost row of harped strands is at a distance of 2 inches from the top fiber of the girder) is calculated as follows (see Fig. A.2.7.2).

$$e_e = 24.75 - \frac{10(2+4+6) + 8(8) + 6(10) + 2(52+50+48+46+44)}{54}$$

$$= 11.34 \text{ in.}$$

Concrete stress at the top fiber of the girder at the girder end at transfer stage:

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_e}{S_t}$$

$$= \frac{1506.58}{788.4} - \frac{1506.58(11.34)}{8902.67} = 1.911 - 1.919 = -0.008 \text{ ksi}$$

Tensile stress limit for fully prestressed concrete members with bonded reinforcement is  $0.24\sqrt{f'_{ci}}$  [LRFD Art. 5.9.4.1]

$$f'_{ci \text{ -reqd.}} = 1000 \left( \frac{0.008}{0.24} \right)^2 = 1.11 \text{ psi}$$

Concrete stress at the bottom fiber of the girder at the girder end at transfer:

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_e}{S_b}$$

$$= \frac{1506.58}{788.4} + \frac{1506.58(11.34)}{10,521.33} = 1.911 + 1.624 = 3.535 \text{ ksi}$$

Compressive stress limit for pretensioned members at transfer is  $0.60 f'_{ci}$ . [LRFD Art. 5.9.4.1]

$$f'_{ci \text{ -reqd.}} = \frac{3535}{0.60} = 5892 \text{ psi (controls)}$$

The required concrete strengths are updated based on the above results as follows.

Concrete strength at release,  $f'_{ci} = 5892 \text{ psi}$

Concrete strength at service,  $f'_c$  is greater of 4973 psi and  $f'_{ci}$

$f'_c = 5892 \text{ psi}$



The difference in the required concrete strengths at release and at service obtained from iterations 2 and 3 is almost 20 psi. Hence, the concrete strengths have sufficiently converged, and another iteration is not required.

Therefore, provide:

$f'_{ci}$  = 5892 psi (as compared to 5455 psi obtained for the Standard design example, an increase of 8 percent)

$f'_c$  = 5892 psi (as compared to 5583 psi obtained for the Standard design example, an increase of 5.5 percent)

54 – 0.5 in. diameter, 10 draped at the end, GR 270 low-relaxation strands (as compared to 50 strands obtained for the Standard design example, an increase of 8 percent)

The final strand patterns at the midspan section and at the girder ends are shown in Figures A.2.7.1 and A.2.7.2. The longitudinal strand profile is shown in Figure A.2.7.3.

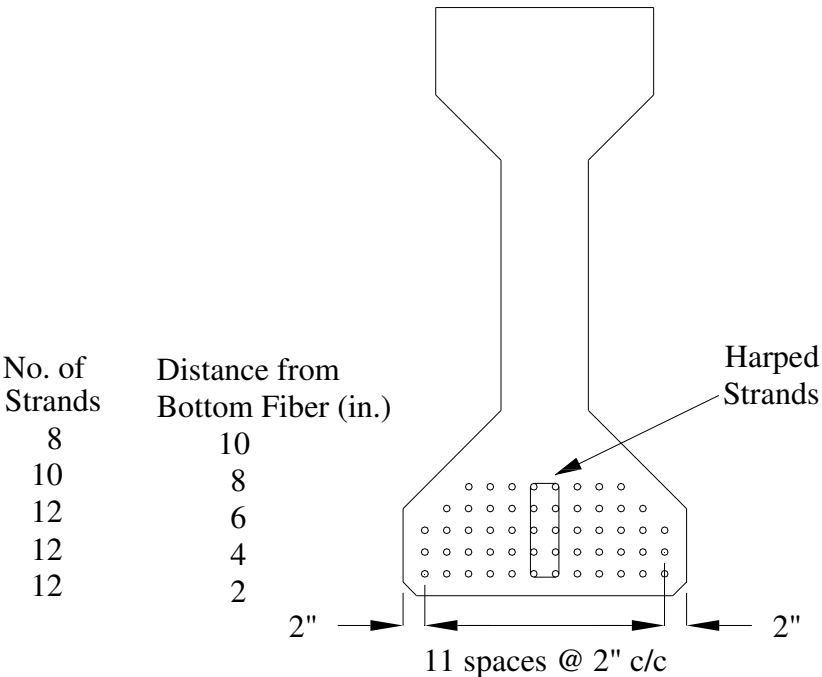


Figure A.2.7.1. Final Strand Pattern at Midspan Section.

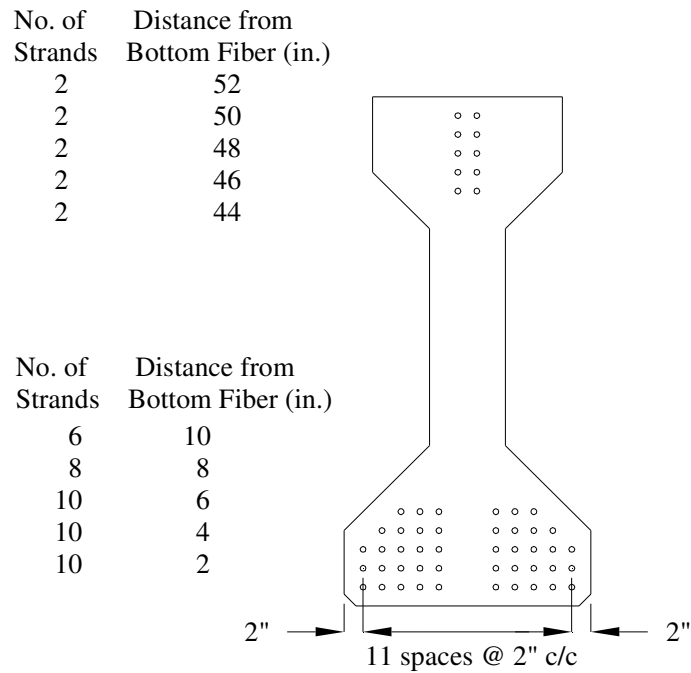


Figure A.2.7.2. Final Strand Pattern at Girder End.

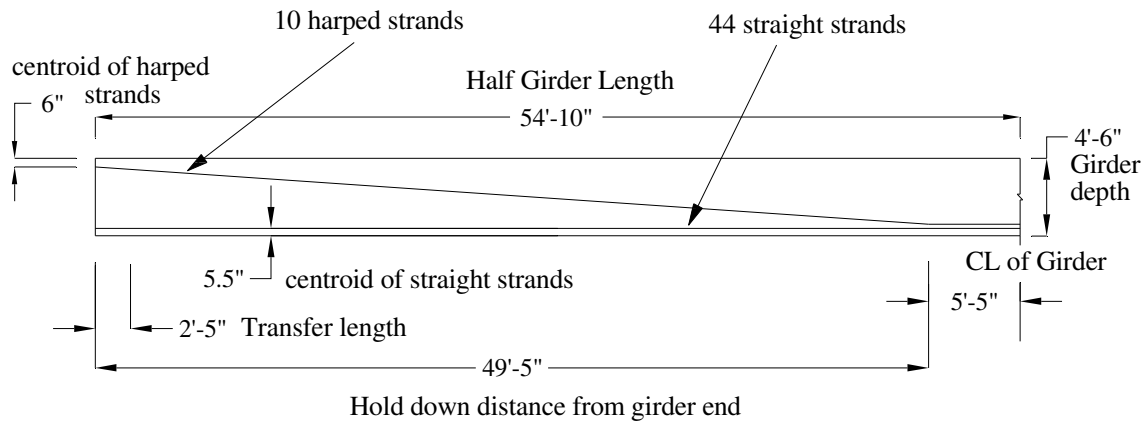


Figure A.2.7.3. Longitudinal Strand Profile (half of the girder length is shown).

The distance between the centroid of the 10 harped strands and the top fiber of the girder the girder end

$$= \frac{2(2) + 2(4) + 2(6) + 2(8) + 2(10)}{10} = 6 \text{ in.}$$

The distance between the centroid of the 10 harped strands and the bottom fiber of the girder at the harp points

$$= \frac{2(2) + 2(4) + 2(6) + 2(8) + 2(10)}{10} = 6 \text{ in.}$$

Transfer length distance from girder end = 60(strand diameter)

[LRFD Art. 5.8.2.3]

Transfer length = 60(0.50) = 30 in. = 2'-6"

The distance between the centroid of 10 harped strands and the top of the girder at the transfer length section

$$= 6 \text{ in.} + \frac{(54 \text{ in.} - 6 \text{ in.} - 6 \text{ in.})}{49.4 \text{ ft.}} (2.5 \text{ ft.}) = 8.13 \text{ in.}$$

The distance between the centroid of the 44 straight strands and the bottom fiber of the girder at all locations

$$= \frac{10(2) + 10(4) + 10(6) + 8(8) + 6(10)}{44} = 5.55 \text{ in.}$$

**A.2.8**  
**STRESS SUMMARY**  
**A.2.8.1**  
**Concrete Stresses at**  
**Transfer**  
**A.2.8.1.1**  
**Allowable Stress**  
**Limits**

[LRFD Art. 5.9.4]

The allowable stress limits at transfer for fully prestressed components, specified by the LRFD Specifications are as follows.

Compression:  $0.6 f'_{ci} = 0.6(5892) = +3535 \text{ psi} = +3.535 \text{ ksi (comp.)}$

Tension: The maximum allowable tensile stress for fully prestressed components is specified as follows:

- In areas other than the precompressed tensile zone and without bonded reinforcement:  $0.0948 \sqrt{f'_{ci}} \leq 0.2 \text{ ksi}$   
 $0.0948 \sqrt{f'_{ci}} = 0.0948 \sqrt{5.892} = 0.23 \text{ ksi} > 0.2 \text{ ksi}$

Allowable tension without bonded reinforcement = - 0.2 ksi

- In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of  $0.5f_y$ , not to exceed 30 ksi (see LRFD C 5.9.4.1.2):

$$0.24\sqrt{f'_{ci}} = 0.24\sqrt{5.892} = -0.582 \text{ ksi (tension)}$$

#### **A.2.8.1.2 Stresses at Girder Ends**

Stresses at the girder ends are checked only at transfer, because it almost always governs.

The eccentricity of the prestressing strands at the girder end when 10 web strands are harped to the topmost location (centroid of the topmost row of harped strands is at a distance of 2 inches from the top fiber of the girder) is calculated as follows (see Fig. A.2.7.2).

$$\begin{aligned} e_e &= 24.75 - \frac{10(2+4+6) + 8(8) + 6(10) + 2(52+50+48+46+44)}{54} \\ &= 11.34 \text{ in.} \end{aligned}$$

Prestressing force after allowing for initial prestress loss

$$\begin{aligned} P_i &= (\text{number of strands})(\text{area of strand})(\text{effective initial prestress}) \\ &= 54(0.153)(182.35) = 1506.58 \text{ kips} \end{aligned}$$

(Effective initial prestress calculations are presented in Section A.2.7.3.5.)

Concrete stress at the top fiber of the girder at the girder end at transfer stage:

$$\begin{aligned} f_{ti} &= \frac{P_i}{A} - \frac{P_i e_e}{S_t} \\ &= \frac{1506.58}{788.4} - \frac{1506.58(11.34)}{8902.67} = 1.911 - 1.919 = -0.008 \text{ ksi} \end{aligned}$$

Allowable tension without additional bonded reinforcement is  $-0.20 \text{ ksi} < -0.008 \text{ ksi (reqd.)}$  (O.K.)

(The additional bonded reinforcement is not required in this case, but where necessary, required area of reinforcement can be calculated using LRFD C 5.9.4.1.2.)

Concrete stress at the bottom fiber of the girder at the girder end at transfer stage:

$$f_{bi} = \frac{P_i}{A} + \frac{P_i e_e}{S_b}$$

$$= \frac{1506.58}{788.4} + \frac{1506.58 (11.34)}{10,521.33} = 1.911 + 1.624 = +3.535 \text{ ksi}$$

Allowable compression: +3.535 ksi = +3.535 ksi (reqd.) (O.K.)

### **A.2.8.1.3** **Stresses at Transfer** **Length Section**

Stresses at transfer length are checked only at release, because it almost always governs.

$$\begin{aligned} \text{Transfer length} &= 60(\text{strand diameter}) && [\text{LRFD Art. 5.8.2.3}] \\ &= 60(0.5) = 30 \text{ in.} = 2'-6" \end{aligned}$$

The transfer length section is located at a distance of 2'-6" from the end of the girder or at a point 1'-11.5" from the centerline of the bearing support, as the girder extends 6.5 in. beyond the bearing centerline. Overall girder length of 109'-8" is considered for the calculation of bending moment at the transfer length section.

$$M_g = 0.5wx(L - x)$$

where:

$$w = \text{Self-weight of the girder} = 0.821 \text{ kips/ft.}$$

$$L = \text{Overall girder length} = 109.67 \text{ ft.}$$

$$x = \text{Transfer length distance from girder end} = 2.5 \text{ ft.}$$

$$M_g = 0.5(0.821)(2.5)(109.67 - 2.5) = 109.98 \text{ k-ft.}$$

Eccentricity of prestressing strands at transfer length section

$$e_t = e_c - (e_c - e_e) \frac{(49.404 - x)}{49.404}$$

where:

$$e_c = \text{Eccentricity of prestressing strands at midspan} = 19.12 \text{ in.}$$

$$e_e = \text{Eccentricity of prestressing strands at girder end} = 11.34 \text{ in.}$$

$$x = \text{Distance of transfer length section from girder end} = 2.5 \text{ ft.}$$

$$e_t = 19.12 - (19.12 - 11.34) \frac{(49.404 - 2.5)}{49.404} = 11.73 \text{ in.}$$

Initial concrete stress at top fiber of the girder at the transfer length section due to self-weight of the girder and effective initial prestress

$$\begin{aligned}
 f_{ti} &= \frac{P_i}{A} - \frac{P_i e_t}{S_t} + \frac{M_g}{S_t} \\
 &= \frac{1506.58}{788.4} - \frac{1506.58(11.73)}{8902.67} + \frac{109.98(12 \text{ in./ft.})}{8902.67} \\
 &= 1.911 - 1.985 + 0.148 = +0.074 \text{ ksi}
 \end{aligned}$$

Allowable compression: +3.535 ksi >> 0.074 ksi (reqd.) (O.K.)

Initial concrete stress at bottom fiber of the girder at hold-down point due to self-weight of girder and effective initial prestress

$$\begin{aligned}
 f_{bi} &= \frac{P_i}{A} + \frac{P_i e_t}{S_b} - \frac{M_g}{S_b} \\
 &= \frac{1506.58}{788.4} + \frac{1506.58(11.73)}{10,521.33} - \frac{109.98(12 \text{ in./ft.})}{10,521.33} \\
 &= 1.911 + 1.680 - 0.125 = 3.466 \text{ ksi}
 \end{aligned}$$

Allowable compression: +3.535 ksi > 3.466 ksi (reqd.) (O.K.)

#### **A.2.8.1.4 Stresses at Hold- Down Points**

The eccentricity of the prestressing strands at the harp points is the same as at midspan.

$$e_{harp} = e_c = 19.12 \text{ in.}$$

Initial concrete stress at top fiber of the girder at hold-down point due to self-weight of the girder and effective initial prestress

$$f_{ti} = \frac{P_i}{A} - \frac{P_i e_{harp}}{S_t} + \frac{M_g}{S_t}$$

where:

$M_g$  = Moment due to girder self-weight at hold-down point based on overall girder length of 109'-8" = 1222.22 k-ft. (see Section A.2.7.1.8)

$$\begin{aligned}
 f_{ti} &= \frac{1506.58}{788.4} - \frac{1506.58(19.12)}{8902.67} + \frac{1222.22(12 \text{ in./ft.})}{8902.67} \\
 &= 1.911 - 3.236 + 1.647 = 0.322 \text{ ksi}
 \end{aligned}$$

Allowable compression: +3.535 ksi >> 0.322 ksi (reqd.) (O.K.)

Initial concrete stress at bottom fiber of the girder at hold-down point due to self-weight of the girder and effective initial prestress

$$\begin{aligned}
 f_{bi} &= \frac{P_i}{A} + \frac{P_i e_{harp}}{S_b} - \frac{M_g}{S_b} \\
 &= \frac{1506.58}{788.4} + \frac{1506.58 (19.12)}{10,521.33} - \frac{1222.22 (12 \text{ in./ft.})}{10,521.33} \\
 &= 1.911 + 2.738 - 1.394 = 3.255 \text{ ksi}
 \end{aligned}$$

Allowable compression: +3.535 ksi > 3.255 ksi (reqd.) (O.K.)

#### **A.2.8.1.5 Stresses at Midspan**

Bending moment due to girder self-weight at midspan section based on overall girder length of 109'-8"

$$M_g = 0.5wx(L - x)$$

where:

$$\begin{aligned}
 w &= \text{Self-weight of the girder} = 0.821 \text{ kips/ft.} \\
 L &= \text{Overall girder length} = 109.67 \text{ ft.} \\
 x &= \text{Half the girder length} = 54.84 \text{ ft.}
 \end{aligned}$$

$$M_g = 0.5(0.821)(54.84)(109.67 - 54.84) = 1234.32 \text{ k-ft.}$$

Initial concrete stress at top fiber of the girder at midspan section due to self-weight of girder and effective initial prestress

$$\begin{aligned}
 f_{ti} &= \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{M_g}{S_t} \\
 &= \frac{1506.58}{788.4} - \frac{1506.58 (19.12)}{8902.67} + \frac{1234.32 (12 \text{ in./ft.})}{8902.67} \\
 &= 1.911 - 3.236 + 1.664 = +0.339 \text{ ksi}
 \end{aligned}$$

Allowable compression: +3.535 ksi >> +0.339 ksi (reqd.) (O.K.)

Initial concrete stress at bottom fiber of the girder at midspan section due to self-weight of the girder and effective initial prestress

$$\begin{aligned}
 f_{bi} &= \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{M_g}{S_b} \\
 &= \frac{1506.58}{788.4} + \frac{1506.58 (19.12)}{10,521.33} - \frac{1234.32 (12 \text{ in./ft.})}{10,521.33} \\
 &= 1.911 + 2.738 - 1.408 = 3.241 \text{ ksi}
 \end{aligned}$$

Allowable compression: +3.535 ksi > 3.241 ksi (reqd.) (O.K.)

**A.2.8.1.6**  
**Stress Summary at**  
**Transfer**

Allowable Stress Limits:

Compression: + 3.535 ksi

Tension: – 0.20 ksi without additional bonded reinforcement

– 0.582 ksi with additional bonded reinforcement

Stresses due to effective initial prestress and self-weight of the girder:

| Location                | Top of girder<br>$f_t$ (ksi) | Bottom of girder<br>$f_b$ (ksi) |
|-------------------------|------------------------------|---------------------------------|
| Girder end              | –0.008                       | +3.535                          |
| Transfer length section | +0.074                       | +3.466                          |
| Hold-down points        | +0.322                       | +3.255                          |
| Midspan                 | +0.339                       | +3.241                          |

**A.2.8.2**  
**Concrete Stresses at**  
**Service Loads**

**A.2.8.2.1**  
**Allowable Stress**  
**Limits**

[LRFD Art. 5.9.4.2]

The allowable stress limits at service load after losses have occurred specified by the LRFD Specifications are presented as follows.

Compression:

Case (I): For stresses due to sum of effective prestress and permanent loads

$$0.45 f'_c = 0.45(5892)/1000 = +2.651 \text{ ksi (for precast girder)}$$

$$0.45 f'_c = 0.45(4000)/1000 = +1.800 \text{ ksi (for slab)}$$

(Note that the allowable stress limit for this case is specified as  $0.40 f'_c$  in Standard Specifications.)

Case (II): For stresses due to live load and one-half the sum of effective prestress and permanent loads

$$0.40 f'_c = 0.40(5892)/1000 = +2.356 \text{ ksi (for precast girder)}$$

$$0.40 f'_c = 0.40(4000)/1000 = +1.600 \text{ ksi (for slab)}$$



Case (III): For stresses due to sum of effective prestress, permanent loads, and transient loads

$$0.60 f'_c = 0.60(5892)/1000 = +3.535 \text{ ksi (for precast girder)}$$

$$0.60 f'_c = 0.60(4000)/1000 = +2.400 \text{ ksi (for slab)}$$

Tension: For components with bonded prestressing tendons that are subjected to not worse than moderate corrosion conditions, for stresses due to load combination Service III

$$0.19 \sqrt{f'_c} = 0.19 \sqrt{5.892} = -0.461 \text{ ksi}$$

#### **A.2.8.2.2** **Final Stresses at** **Midspan**

Effective prestressing force after allowing for final prestress loss

$$P_{pe} = (\text{number of strands})(\text{area of each strand})(f_{pe})$$

$$= 54(0.153)(145.80) = 1204.60 \text{ kips}$$

(Calculations for effective final prestress ( $f_{pe}$ ) are shown in Section A.2.7.3.6.)

Concrete stresses at the top fiber of the girder at the midspan section due to transient loads, permanent loads, and effective final prestress will be investigated for the following three cases using Service I limit state shown as follows.

Case (I): Concrete stress at the top fiber of the girder at the midspan section due to the sum of effective final prestress and permanent loads

$$f_{tf} = \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW}}{S_{tg}}$$

where:

$f_{tf}$  = Concrete stress at the top fiber of the girder, ksi

$M_{DCN}$  = Moment due to non-composite dead loads, k-ft.  
=  $M_g + M_s$

$M_g$  = Moment due to girder self-weight = 1209.98 k-ft.

$M_s$  = Moment due to slab weight = 1179.03 k-ft.

$$M_{DCN} = 1209.98 + 1179.03 = 2389.01 \text{ k-ft.}$$

$$M_{DCC} = \text{Moment due to composite dead loads except wearing surface load, k-ft.} = M_{barr}$$

$$M_{barr} = \text{Moment due to barrier weight} = 160.64 \text{ k-ft.}$$

$$M_{DCC} = 160.64 \text{ k-ft.}$$

$$M_{DW} = \text{Moment due to wearing surface load} = 188.64 \text{ k-ft.}$$

$$S_t = \text{Section modulus referenced to the extreme top fiber of the non-composite precast girder} = 8902.67 \text{ in.}^3$$

$$S_{tg} = \text{Section modulus of composite section referenced to the top fiber of the precast girder} = 54,083.9 \text{ in.}^3$$

$$\begin{aligned} f_{tf} &= \frac{1204.60}{788.4} - \frac{1204.60(19.12)}{8902.67} + \frac{(2389.01)(12 \text{ in./ft.})}{8902.67} \\ &\quad + \frac{(160.64 + 188.64)(12 \text{ in./ft.})}{54,083.9} \\ &= 1.528 - 2.587 + 3.220 + 0.077 = +2.238 \text{ ksi} \end{aligned}$$

$$\text{Allowable compression: } +2.651 \text{ ksi} > +2.238 \text{ ksi (reqd.)} \quad (\text{O.K.})$$

Case (II): Concrete stress at the top fiber of the girder at the midspan section due to the live load and one-half the sum of effective final prestress and permanent loads

$$f_{tf} = \frac{(M_{LT} + M_{LL})}{S_{tg}} + 0.5 \left( \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW}}{S_{tg}} \right)$$

where:

$$M_{LT} = \text{Distributed moment due to HS 20-44 truck load including dynamic load allowance} = 1423.00 \text{ k-ft.}$$

$$M_{LL} = \text{Distributed moment due to lane load} = 602.72 \text{ k-ft.}$$

$$\begin{aligned} f_{tf} &= \frac{(1423.00 + 602.72)(12 \text{ in./ft.})}{54,083.9} + 0.5 \left\{ \frac{1204.60}{788.4} - \frac{1204.60(19.12)}{8902.67} \right. \\ &\quad \left. + \frac{(2389.01)(12 \text{ in./ft.})}{8902.67} + \frac{(160.64 + 188.64)(12 \text{ in./ft.})}{54,083.9} \right\} \\ &= 0.449 + 0.5(1.528 - 2.587 + 3.220 + 0.077) = 1.568 \text{ ksi} \end{aligned}$$

$$\text{Allowable compression: } +2.356 \text{ ksi} > +1.568 \text{ ksi (reqd.)} \quad (\text{O.K.})$$

Case (III): Concrete stress at the top fiber of the girder at the midspan section due to the sum of effective final prestress, permanent loads, and transient loads

$$\begin{aligned}
 f_{tf} &= \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{M_{DCN}}{S_t} + \frac{M_{DCC} + M_{DW} + M_{LT} + M_{LL}}{S_{tg}} \\
 &= \frac{1204.60}{788.4} - \frac{1204.60(19.12)}{8902.67} + \frac{(2389.01)(12 \text{ in./ft.})}{8902.67} \\
 &\quad + \frac{(160.64 + 188.64 + 1423.00 + 602.72)(12 \text{ in./ft.})}{54,083.9} \\
 &= 1.528 - 2.587 + 3.220 + 0.527 = 2.688 \text{ ksi}
 \end{aligned}$$

Allowable compression: +3.535 ksi > 2.688 ksi (reqd.) (O.K.)

Concrete stresses at the bottom fiber of the girder at the midspan section due to transient loads, permanent loads, and effective final prestress is investigated using Service III limit state as follows.

$$f_{bf} = \frac{P_{pe}}{A} + \frac{P_{pe} e_c}{S_b} - \frac{M_{DCN}}{S_b} - \frac{M_{DCC} + M_{DW} + 0.8(M_{LT} + M_{LL})}{S_{bc}}$$

where:

$S_b$  = Section modulus referenced to the extreme bottom fiber of the non-composite precast girder = 10,521.33 in.<sup>3</sup>

$S_{bc}$  = Section modulus of composite section referenced to the extreme bottom fiber of the precast girder  
= 16,876.83 in.<sup>3</sup>

$$\begin{aligned}
 f_{bf} &= \frac{1204.60}{788.4} + \frac{1204.60(19.12)}{10,521.33} - \frac{(2389.01)(12 \text{ in./ft.})}{10,521.33} \\
 &\quad - \frac{[160.64 + 188.64 + 0.8(1423.00 + 602.72)](12 \text{ in./ft.})}{16,876.83} \\
 &= 1.528 + 2.189 - 2.725 - 1.401 = -0.409 \text{ ksi}
 \end{aligned}$$

Allowable tension: -0.461 ksi < -0.409 ksi (reqd.) (O.K.)

Superimposed dead loads and live loads contribute to the stresses at the top of the slab calculated as follows.

Case (I): Superimposed dead load effect

Concrete stress at the top fiber of the slab at midspan section due to superimposed dead loads

$$\begin{aligned} f_t &= \frac{M_{DCC} + M_{DW}}{S_{tc}} \\ &= \frac{(160.64 + 188.64)(12 \text{ in./ft.})}{33,325.31} = 0.126 \text{ ksi} \end{aligned}$$

Allowable compression: +1.800 ksi >> +0.126 ksi (reqd.) (O.K.)

Case (II): Live load + 0.5(superimposed dead loads)

Concrete stress at the top fiber of the slab at midspan section due to sum of live loads and one-half the superimposed dead loads

$$\begin{aligned} f_t &= \frac{M_{LT} + M_{LL} + 0.5(M_{DCC} + M_{DW})}{S_{tc}} \\ &= \frac{[1423.00 + 602.72 + 0.5(160.64 + 188.64)](12 \text{ in./ft.})}{33,325.31} \\ &= +0.792 \text{ ksi} \end{aligned}$$

Allowable compression: +1.600 ksi > +0.792 ksi (reqd.) (O.K.)

Case (III): Superimposed dead loads + Live load

Concrete stress at the top fiber of the slab at midspan section due to sum of permanent loads and live load.

$$\begin{aligned} f_t &= \frac{M_{LT} + M_{LL} + M_{DCC} + M_{DW}}{S_{tc}} \\ &= \frac{[1423.00 + 602.72 + 160.64 + 188.64](12 \text{ in./ft.})}{33,325.31} = +0.855 \text{ ksi} \end{aligned}$$

Allowable compression: +2.400 ksi > +0.855 ksi (reqd.) (O.K.)

### **A.2.8.2.3** **Summary of Stresses** **at Service Loads**

The final stresses at the top and bottom fiber of the girder and at the top fiber of the slab at service conditions for the cases defined in Section A.2.8.2.2 are summarized as follows.

| At Midspan | Top of slab<br>$f_t$ (ksi) | Top of Girder<br>$f_t$ (ksi) | Bottom of girder<br>$f_b$ (ksi) |
|------------|----------------------------|------------------------------|---------------------------------|
| Case I     | +0.126                     | +2.238                       | –                               |
| Case II    | +0.792                     | +1.568                       | –                               |
| Case III   | +0.855                     | +2.688                       | – 0.409                         |

### **A.2.8.2.4** **Composite Section** **Properties**

The composite section properties calculated in Section A.2.4.2.3 were based on the modular ratio value of 1. But as the actual concrete strength is now selected, the actual modular ratio can be determined and the corresponding composite section properties can be evaluated. The updated composite section properties are presented in Table A.2.8.1.

Modular ratio between slab and girder concrete

$$n = \left( \frac{E_{cs}}{E_{cp}} \right)$$

where:

$n$  = Modular ratio between slab and girder concrete

$E_{cs}$  = Modulus of elasticity of slab concrete, ksi  
 $= 33,000(w_c)^{1.5} \sqrt{f'_{cs}}$  [LRFD Eq. 5.4.2.4-1]

$w_c$  = Unit weight of concrete = (must be between 0.09 and 0.155 kcf for LRFD Eq. 5.4.2.4-1 to be applicable)  
 $= 0.150$  kcf

$f'_{cs}$  = Compressive strength of slab concrete at service  
 $= 4.0$  ksi

$E_{cs} = [33,000(0.150)^{1.5} \sqrt{4}] = 3834.25$  ksi

$E_{cp}$  = Modulus of elasticity of girder concrete at service, ksi  
 $= 33,000(w_c)^{1.5} \sqrt{f'_c}$

$f'_c$  = Compressive strength of precast girder concrete at service  
 $= 5.892$  ksi

$$E_{cp} = [33,000(0.150)^{1.5} \sqrt{5.892}] = 4653.53 \text{ ksi}$$

$$n = \frac{3834.25}{4653.53} = 0.824$$

Transformed flange width,  $b_{tf} = n \times (\text{effective flange width})$

Effective flange width = 96 in. (see Section A.2.4.2)

$$b_{tf} = 0.824(96) = 79.10 \text{ in.}$$

Transformed Flange Area,  $A_{tf} = n \times (\text{effective flange width})(t_s)$

$t_s$  = Slab thickness = 8 in.

$$A_{tf} = 0.824(96)(8) = 632.83 \text{ in.}^2$$

*Table A.2.8.1. Properties of Composite Section.*

|          | Transformed Area<br>$A \text{ (in.}^2\text{)}$ | $y_b$<br>in. | $Ay_b$<br>in. <sup>3</sup> | $A(y_{bc} - y_b)^2$ | $I$<br>in. <sup>4</sup> | $I + A(y_{bc} - y_b)^2$<br>in. <sup>4</sup> |
|----------|--|--------------|----------------------------|---------------------|-------------------------|---|
| Girder   | 788.40   | 24.75        | 19,512.9                   | 172,924.58          | 260,403.0               | 433,327.6                                   |
| Slab     | 632.83   | 58.00        | 36,704.1                   | 215,183.46          | 3,374.9                 | 218,558.4                                   |
| $\Sigma$ | 1,421.23                                       |              | 56,217.0                   |                     |                         | 651,886.0                                   |

$$A_c = \text{Total area of composite section} = 1421.23 \text{ in.}^2$$

$$h_c = \text{Total height of composite section} = 54 \text{ in.} + 8 \text{ in.} = 62 \text{ in.}$$

$$I_c = \text{Moment of inertia of composite section} = 651,886.0 \text{ in.}^4$$

$$y_{bc} = \text{Distance from the centroid of the composite section to extreme bottom fiber of the precast girder, in.} \\ = 56,217.0/1421.23 = 39.56 \text{ in.}$$

$$y_{tg} = \text{Distance from the centroid of the composite section to extreme top fiber of the precast girder, in.} \\ = 54 - 39.56 = 14.44 \text{ in.}$$

$$y_{tc} = \text{Distance from the centroid of the composite section to extreme top fiber of the slab} = 62 - 39.56 = 22.44 \text{ in.}$$

$$S_{bc} = \text{Section modulus of composite section referenced to the extreme bottom fiber of the precast girder, in.}^3 \\ = I_c/y_{bc} = 651,886.0/39.56 = 16,478.41 \text{ in.}^3$$

$$S_{tg} = \text{Section modulus of composite section referenced to the top fiber of the precast girder, in.}^3 \\ = I_c/y_{tg} = 651,886.0/14.44 = 45,144.46 \text{ in.}^3$$

$$S_{tc} = \text{Section modulus of composite section referenced to the top fiber of the slab, in.}^3 \\ = I_c/y_{tc} = 651,886.0/22.44 = 29,050.18 \text{ in.}^3$$

### **A.2.9 CHECK FOR LIVE LOAD MOMENT DISTRIBUTION FACTOR**

The live load moment distribution factor calculation involves a parameter for longitudinal stiffness,  $K_g$ . This parameter depends on the modular ratio between the girder and the slab concrete. The live load moment distribution factor calculated in Section A.2.5.2.2.1 is based on the assumption that the modular ratio between the girder and slab concrete is 1. However, as the actual concrete strength is now chosen, the live load moment distribution factor based on the actual modular ratio needs to be calculated and compared to the distribution factor calculated in Section A.2.5.2.2.1. If the difference between the two is found to be large, the bending moments have to be updated based on the calculated live load moment distribution factor.

$$K_g = n(I + A e_g^2) \quad [\text{LRFD Art. 3.6.1.1.1}]$$

where:

$$n = \text{Modular ratio between girder and slab concrete} \\ = \frac{E_c \text{ for girder concrete}}{E_c \text{ for slab concrete}} = \left( \frac{E_{cp}}{E_{cs}} \right)$$

(Note that this ratio is the inverse of the one defined for composite section properties in Section A.2.8.2.4.)

$$E_{cs} = \text{Modulus of elasticity of slab concrete, ksi} \\ = 33,000(w_c)^{1.5} \sqrt{f'_{cs}} \quad [\text{LRFD Eq. 5.4.2.4-1}]$$

$$w_c = \text{Unit weight of concrete} = (\text{must be between 0.09 and 0.155 kcf for LRFD Eq. 5.4.2.4-1 to be applicable}) \\ = 0.150 \text{ kcf}$$

$$f'_{cs} = \text{Compressive strength of slab concrete at service} \\ = 4.0 \text{ ksi}$$

$$E_{cs} = [33,000(0.150)^{1.5} \sqrt{4}] = 3834.25 \text{ ksi}$$

$$E_{cp} = \text{Modulus of elasticity of girder concrete at service, ksi} \\ = 33,000(w_c)^{1.5} \sqrt{f'_c}$$

$$f'_c = \text{Compressive strength of precast girder concrete at service} \\ = 5.892 \text{ ksi}$$

$$E_{cp} = [33,000(0.150)^{1.5} \sqrt{5.892}] = 4653.53 \text{ ksi}$$

$$n = \frac{4653.53}{3834.25} = 1.214$$

$$A = \text{Area of girder cross section (non-composite section)} \\ = 788.4 \text{ in.}^2$$

$$I = \text{Moment of inertia about the centroid of the non-composite precast girder} = 260,403 \text{ in.}^4$$

$$e_g = \text{Distance between centers of gravity of the girder and slab, in.} \\ = (t_s/2 + y_t) = (8/2 + 29.25) = 33.25 \text{ in.}$$

$$K_g = (1.214)[260,403 + 788.4 (33.25)^2] = 1,374,282.6 \text{ in.}^4$$

The approximate live load moment distribution factors for type k bridge girders, specified by LRFD Table 4.6.2.2b-1 are applicable if the following condition for  $K_g$  is satisfied (other requirements are provided in section A.2.5.2.2.1).

$$10,000 \leq K_g \leq 7,000,000$$

$$10,000 \leq 1,374,282.6 \leq 7,000,000 \quad (\text{O.K.})$$

For one design lane loaded:

$$DFM = 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12.0 L t_s^3}\right)^{0.1}$$

where:

$$DFM = \text{Live load moment distribution factor for interior girders}$$

$$S = \text{Spacing of adjacent girders} = 8 \text{ ft.}$$

$$L = \text{Design span length} = 108.583 \text{ ft.}$$

$$t_s = \text{Thickness of slab} = 8 \text{ in.}$$



$$DFM = 0.06 + \left(\frac{8}{14}\right)^{0.4} \left(\frac{8}{108.583}\right)^{0.3} \left(\frac{1,374,282.6}{12.0(108.583)(8)^3}\right)^{0.1}$$

$$DFM = 0.06 + (0.8)(0.457)(1.075) = 0.453 \text{ lanes/girder}$$

For two or more lanes loaded:

$$DFM = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12.0 L t_s^3}\right)^{0.1}$$

$$DFM = 0.075 + \left(\frac{8}{9.5}\right)^{0.6} \left(\frac{8}{108.583}\right)^{0.2} \left(\frac{1,374,282.6}{12.0(108.583)(8)^3}\right)^{0.1}$$

$$= 0.075 + (0.902)(0.593)(1.075) = 0.650 \text{ lanes/girder}$$

The greater of the above two distribution factors governs. Thus, the case of two or more lanes loaded controls.

$$DFM = 0.650 \text{ lanes/girder}$$

The live load moment distribution factor from Section A.2.5.2.2.1 is  $DFM = 0.639$  lanes/girder.

$$\text{Percent difference in } DFM = \left(\frac{0.650 - 0.639}{0.650}\right)100 = 1.69 \text{ percent}$$

The difference in the live load moment distribution factors is negligible, and its impact on the live load moments will also be negligible. Hence, the live load moments obtained using the distribution factor from Section A.2.5.2.2.1 can be used for the ultimate flexural strength design.

## **A.2.10 FATIGUE LIMIT STATE**

LRFD Art. 5.5.3 specifies that the check for fatigue of the prestressing strands is not required for fully prestressed components that are designed to have extreme fiber tensile stress due to the Service III limit state within the specified limit of  $0.19\sqrt{f'_c}$ .

The AASHTO Type IV girder in this design example is designed as a fully prestressed member, and the tensile stress due to Service III limit state is less than  $0.19\sqrt{f'_c}$ , as shown in Section A.2.8.2.2. Hence, the fatigue check for the prestressing strands is not required.

**A.2.11**  
**FLEXURAL STRENGTH**  
**LIMIT STATE**

[LRFD Art. 5.7.3]

The flexural strength limit state is investigated for the Strength I load combination specified by LRFD Table 3.4.1-1 as follows.

$$M_u = 1.25(M_{DC}) + 1.5(M_{DW}) + 1.75(M_{LL+IM})$$

where:

$M_u$  = Factored ultimate moment at the midspan, k-ft.

$M_{DC}$  = Moment at the midspan due to dead load of structural components and non-structural attachments, k-ft.  
 $= M_g + M_S + M_{barr}$

$M_g$  = Moment at the midspan due to girder self-weight  
 $= 1209.98$  k-ft.

$M_S$  = Moment at the midspan due to slab weight  
 $= 1179.03$  k-ft.

$M_{barr}$  = Moment at the midspan due to barrier weight  
 $= 160.64$  k-ft.

$M_{DC}$  =  $1209.98 + 1179.03 + 160.64 = 2549.65$  k-ft.

$M_{DW}$  = Moment at the midspan due to wearing surface load  
 $= 188.64$  k-ft.

$M_{LL+IM}$  = Moment at the midspan due to vehicular live load including dynamic allowance, k-ft.  
 $= M_{LT} + M_{LL}$

$M_{LT}$  = Distributed moment due to HS 20-44 truck load including dynamic load allowance =  $1423.00$  k-ft.

$M_{LL}$  = Distributed moment due to lane load =  $602.72$  k-ft.

$M_{LL+IM}$  =  $1423.00 + 602.72 = 2025.72$  k-ft.

The factored ultimate bending moment at midspan

$$\begin{aligned} M_u &= 1.25(2549.65) + 1.5(188.64) + 1.75(2025.72) \\ &= 7015.03 \text{ k-ft.} \end{aligned}$$

[LRFD Art. 5.7.3.1.1]

The average stress in the prestressing steel,  $f_{ps}$ , for rectangular or flanged sections subjected to flexure about one axis for which  $f_{pe} \geq 0.5f_{pu}$ , is given as:

$$f_{ps} = f_{pu} \left( 1 - k \frac{c}{d_p} \right) \quad [\text{LRFD Eq. 5.7.3.1.1-1}]$$

where:

$f_{ps}$  = Average stress in the prestressing steel, ksi

$f_{pu}$  = Specified tensile strength of prestressing steel = 270 ksi

$f_{pe}$  = Effective prestress after final losses =  $f_{pj} - \Delta f_{pT}$

$f_{pj}$  = Jacking stress in the prestressing strands = 202.5 ksi

$\Delta f_{pT}$  = Total final loss in prestress = 56.70 ksi (Section A.2.7.3.6)

$f_{pe} = 202.5 - 56.70 = 145.80 \text{ ksi} > 0.5f_{pu} = 0.5(270) = 135 \text{ ksi}$   
Therefore, the equation for  $f_{ps}$  shown above is applicable.

$$k = 2 \left( 1.04 - \frac{f_{py}}{f_{pu}} \right) \quad [\text{LRFD Eq. 5.7.3.1.1-2}]$$

= 0.28 for low-relaxation prestressing strands

[LRFD Table C5.7.3.1.1-1]

$d_p$  = Distance from the extreme compression fiber to the centroid of the prestressing tendons, in.  
=  $h_c - y_{bs}$

$h_c$  = Total height of the composite section =  $54 + 8 = 62 \text{ in.}$

$y_{bs}$  = Distance from centroid of the prestressing strands at midspan to the bottom fiber of the girder = 5.63 in. (see Section A.2.7.3.3)

$d_p = 62 - 5.63 = 56.37 \text{ in.}$

$c$  = Distance between neutral axis and the compressive face of the section, in.

The depth of the neutral axis from the compressive face,  $c$ , is computed assuming rectangular section behavior. A check is made to confirm that the neutral axis is lying in the cast-in-place slab; otherwise, the neutral axis will be calculated based on the flanged section behavior. [LRFD C5.7.3.2.2]

For rectangular section behavior,

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A'_s f'_s}{0.85 f'_c \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}} \quad [\text{LRFD Eq. 5.7.3.1.1.-4}]$$

$$\begin{aligned} A_{ps} &= \text{Area of prestressing steel, in.}^2 \\ &= (\text{number of strands})(\text{area of each strand}) \\ &= 54(0.153) = 8.262 \text{ in.}^2 \end{aligned}$$

$$f_{pu} = \text{Specified tensile strength of prestressing steel} = 270 \text{ ksi}$$

$$A_s = \text{Area of mild steel tension reinforcement} = 0 \text{ in.}^2$$

$$A'_s = \text{Area of compression reinforcement} = 0 \text{ in.}^2$$

$$f'_c = \text{Compressive strength of deck concrete} = 4.0 \text{ ksi}$$

$$f_y = \text{Yield strength of tension reinforcement, ksi}$$

$$f'_y = \text{Yield strength of compression reinforcement, ksi}$$

$$\begin{aligned} \beta_1 &= \text{Stress factor for compression block} \quad [\text{LRFD Art. 5.7.2.2}] \\ &= 0.85 \text{ for } f'_c \leq 4.0 \text{ ksi} \end{aligned}$$

$$b = \text{Effective width of compression flange} = 96 \text{ in. (based on non-transformed section)}$$

Depth of neutral axis from compressive face

$$\begin{aligned} c &= \frac{8.262(270) + 0 - 0}{0.85(4.0)(0.85)(96) + 0.28(8.262)\left(\frac{270}{56.37}\right)} \\ &= 7.73 \text{ in.} < t_s = 8.0 \text{ in. (O.K.)} \end{aligned}$$

The neutral axis lies in the slab; therefore, the assumption of rectangular section behavior is valid.

The average stress in prestressing steel

$$f_{ps} = 270 \left( 1 - 0.28 \frac{7.73}{56.37} \right) = 259.63 \text{ ksi}$$

For prestressed concrete members having rectangular section behavior, the nominal flexural resistance is given as:

[LRFD Art. 5.7.3.2.3]

$$M_n = A_{ps} f_{ps} \left( d_p - \frac{a}{2} \right) \quad [\text{LRFD Eq. 5.7.3.2.2-1}]$$

The above equation is a simplified form of LRFD Equation 5.7.3.2.2-1 because no compression reinforcement or mild tension reinforcement is provided.

$a$  = Depth of the equivalent rectangular compression block, in.  
 $= \beta_1 c$

$\beta_1$  = Stress factor for compression block = 0.85 for  $f'_c \leq 4.0$  ksi

$a = 0.85(7.73) = 6.57$  in.

Nominal flexural resistance

$$\begin{aligned} M_n &= (8.262)(259.63) \left( 56.37 - \frac{6.57}{2} \right) \\ &= 113,870.67 \text{ k-in.} = 9,489.22 \text{ k-ft.} \end{aligned}$$

Factored flexural resistance:

$$M_r = \phi M_n \quad [\text{LRFD Eq. 5.7.3.2.1-1}]$$

where:

$\phi$  = Resistance factor [LRFD Art. 5.5.4.2.1]  
 $= 1.0$  for flexure and tension of prestressed concrete members

$$M_r = 1 \times (9,489.22) = 9,489.22 \text{ k-ft.} > M_u = 7,015.03 \text{ k-ft.} \quad (\text{O.K.})$$

## **A.2.12**

### **LIMITS FOR**

### **REINFORCEMENT**

#### **A.2.12.1**

#### **Maximum**

#### **Reinforcement**

[LRFD Art. 5.7.3.3]

[LRFD Art. 5.7.3.3.1]

The maximum amount of the prestressed and non-prestressed reinforcement should be such that

$$\frac{c}{d_e} \leq 0.42 \quad [\text{LRFD Eq. 5.7.3.3.1-1}]$$

in which:

$$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y} \quad [\text{LRFD Eq. 5.7.3.3.1-2}]$$

$c$  = Distance from the extreme compression fiber to the neutral axis = 7.73 in.

$d_e$  = The corresponding effective depth from the extreme fiber to the centroid of the tensile force in the tensile reinforcement, in.  
 =  $d_p$ , if mild steel tension reinforcement is not used

$d_p$  = Distance from the extreme compression fiber to the centroid of the prestressing tendons = 56.37 in.

Therefore  $d_e = 56.37$  in.

$$\frac{c}{d_e} = \frac{7.73}{56.37} = 0.137 \ll 0.42 \quad (\text{O.K.})$$

### **A.2.12.2 Minimum Reinforcement**

[LRFD Art. 5.7.3.3.2]

At any section of a flexural component, the amount of prestressed and non-prestressed tensile reinforcement should be adequate to develop a factored flexural resistance,  $M_r$ , at least equal to the lesser of:

- 1.2 times the cracking moment,  $M_{cr}$ , determined on the basis of elastic stress distribution and the modulus of rupture of concrete,  $f_r$
- 1.33 times the factored moment required by the applicable strength load combination

The above requirements are checked at the midspan section in this design example. Similar calculations can be performed at any section along the girder span to check these requirements.

The cracking moment is given as

$$M_{cr} = S_c (f_r + f_{cpe}) - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \leq S_c f_r \quad [\text{LRFD Eq. 5.7.3.3.2-1}]$$

where:

$f_r$  = Modulus of rupture, ksi  
 =  $0.24 \sqrt{f'_c}$  for normal weight concrete [LRFD Art. 5.4.2.6]

$f'_c$  = Compressive strength of girder concrete at service  
 = 5.892 ksi

$f_r$  =  $0.24 \sqrt{5.892} = 0.582$  ksi

$f_{cpe}$  = Compressive stress in concrete due to effective prestress force at extreme fiber of the section where tensile stress is caused by externally applied loads, ksi

$$= \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b}$$

$P_{pe}$  = Effective prestressing force after allowing for final prestress loss, kips

$$= (\text{number of strands})(\text{area of each strand})(f_{pe})$$

$$= 54(0.153)(145.80) = 1204.60 \text{ kips}$$

(Calculations for effective final prestress ( $f_{pe}$ ) are shown in Section A.2.7.3.6.)

$e_c$  = Eccentricity of prestressing strands at the midspan  
= 19.12 in.

$A$  = Area of girder cross-section = 788.4 in.<sup>2</sup>

$S_b$  = Section modulus of the precast girder referenced to the extreme bottom fiber of the non-composite precast girder  
= 10,521.33 in.<sup>3</sup>

$$\begin{aligned} f_{cpe} &= \frac{1204.60}{788.4} + \frac{1204.60(19.12)}{10,521.33} \\ &= 1.528 + 2.189 = 3.717 \text{ ksi} \end{aligned}$$

$M_{dnc}$  = Total unfactored dead load moment acting on the non-composite section  
=  $M_g + M_s$

$M_g$  = Moment at the midspan due to girder self-weight  
= 1209.98 k-ft.

$M_s$  = Moment at the midspan due to slab weight  
= 1179.03 k-ft.

$$M_{dnc} = 1209.98 + 1179.03 = 2389.01 \text{ k-ft.} = 28,668.12 \text{ k-in.}$$

$S_{nc}$  = Section modulus of the non-composite section referenced to the extreme fiber where the tensile stress is caused by externally applied loads = 10,521.33 in.<sup>3</sup>

$S_c$  = Section modulus of the composite section referenced to the extreme fiber where the tensile stress is caused by externally applied loads = 16,478.41 in.<sup>3</sup> (based on updated composite section properties)

The cracking moment is:

$$M_{cr} = (16,478.41)(0.582 + 3.717) - (28,668.12) \left( \frac{16,478.41}{10,521.33} - 1 \right)$$

$$= 70,840.68 - 16,231.62 = 54,609.06 \text{ k-in.} = 4,550.76 \text{ k-ft.}$$

$$S_c f_r = (16,478.41)(0.582) = 9,590.43 \text{ k-in.}$$

$$= 799.20 \text{ k-ft.} < 4,550.76 \text{ k-ft.}$$

Therefore, use  $M_{cr} = 799.20 \text{ k-ft.}$

$$1.2 M_{cr} = 1.2(799.20) = 959.04 \text{ k-ft.}$$

Factored moment required by Strength I load combination at midspan

$$M_u = 7015.03 \text{ k-ft.}$$

$$1.33 M_u = 1.33(7,015.03 \text{ k-ft.}) = 9330 \text{ k-ft.}$$

Since,  $1.2 M_{cr} < 1.33 M_u$ , the  $1.2 M_{cr}$  requirement controls.

$$M_r = 9489.22 \text{ k-ft} >> 1.2 M_{cr} = 959.04 \text{ (O.K.)}$$

### **A.2.13 TRANSVERSE SHEAR DESIGN**

The area and spacing of shear reinforcement must be determined at regular intervals along the entire span length of the girder. In this design example, transverse shear design procedures are demonstrated below by determining these values at the critical section near the supports. Similar calculations can be performed to determine shear reinforcement requirements at any selected section.

LRFD Art. 5.8.2.4 specifies that the transverse shear reinforcement is required when:

$$V_u < 0.5 \phi (V_c + V_p) \quad [\text{LRFD Art. 5.8.2.4-1}]$$

where:

$V_u$  = Total factored shear force at the section, kips

$V_c$  = Nominal shear resistance of the concrete, kips

$V_p$  = Component of the effective prestressing force in the direction of the applied shear, kips

$\phi$  = Resistance factor = 0.90 for shear in prestressed concrete members [LRFD Art. 5.5.4.2.1]



### **A.2.13.1 Critical Section**

Critical section near the supports is the greater of:

[LRFD Art. 5.8.3.2]

$$0.5 d_v \cot \theta \text{ or } d_v$$

where:

$d_v$  = Effective shear depth, in.  
 = Distance between the resultants of tensile and compressive forces,  $(d_e - a/2)$ , but not less than the greater of  $(0.9d_e)$  or  $(0.72h)$  [LRFD Art. 5.8.2.9]

$d_e$  = Corresponding effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement [LRFD Art. 5.7.3.3.1]

$a$  = Depth of compression block = 6.57 in. at midspan (see Section A.2.11)

$h$  = Height of composite section = 62 in.

### **A.2.13.1.1 Angle of Diagonal Compressive Stresses**

The angle of inclination of the diagonal compressive stresses is calculated using an iterative method. As an initial estimate  $\theta$  is taken as  $23^\circ$ .

### **A.2.13.1.2 Effective Shear Depth**

The shear design at any section depends on the angle of diagonal compressive stresses at the section. Shear design is an iterative process that begins with assuming a value for  $\theta$ .

Because some of the strands are harped at the girder end, the effective depth  $d_e$ , varies from point to point. However,  $d_e$  must be calculated at the critical section for shear, which is not yet known. Therefore, for the first iteration,  $d_e$  is calculated based on the center of gravity of the straight strand group at the end of the girder,  $y_{bsend}$ . This methodology is given in *PCI Bridge Design Manual* (PCI 2003).

Effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement

$$d_e = h - y_{bsend} = 62.0 - 5.55 = 56.45 \text{ in. (see Section A.2.7.3.9 for } y_{bsend})$$

Effective shear depth

$$d_v = d_e - 0.5(a) = 56.45 - 0.5(6.57) = 53.17 \text{ in.} \quad (\text{controls})$$

$$\geq 0.9d_e = 0.9(56.45) = 50.80 \text{ in.}$$

$$\geq 0.72h = 0.72(62) = 44.64 \text{ in.} \quad (\text{O.K.})$$

Therefore  $d_v = 53.17 \text{ in.}$

### **A.2.13.1.3** **Calculation of** **Critical Section**

[LRFD Art. 5.8.3.2]

The critical section near the support is greater of:

$$d_v = 53.17 \text{ in.} \quad \text{and}$$

$$0.5 d_v \cot \theta = 0.5(53.17)(\cot 23^\circ) = 62.63 \text{ in. from the face of the support} \quad (\text{controls})$$

Adding half the bearing width (3.5 in., standard pad size for prestressed girders is 7" × 22") to the critical section distance from the face of the support to get the distance of the critical section from the centerline of bearing.

Critical section for shear

$$x = 62.63 + 3.5 = 66.13 \text{ in.} = 5.51 \text{ ft. } (0.051L) \text{ from the centerline of the bearing, where } L \text{ is the design span length.}$$

The value of  $d_e$  is calculated at the girder end, which can be refined based on the critical section location. However, it is conservative not to refine the value of  $d_e$  based on the critical section  $0.051L$ . The value, if refined, will have a small difference (PCI 2003).

### **A.2.13.2** **Contribution of** **Concrete to Nominal** **Shear Resistance**

[LRFD Art. 5.8.3.3]

The contribution of the concrete to the nominal shear resistance is given as:

$$V_c = 0.0316\beta\sqrt{f'_c} b_v d_v \quad [\text{LRFD Eq. 5.8.3.3-3}]$$

where:

$\beta$  = A factor indicating the ability of diagonally cracked concrete to transmit tension

$f'_c$  = Compressive strength of concrete at service = 5.892 ksi

$b_v$  = effective web width taken as the minimum web width within the depth  $d_v$ , in. = 8 in. (see Figure A.2.4.1)

$d_v$  = Effective shear depth = 53.17 in.

**A.2.13.2.1**  
**Strain in Flexural**  
**Tension**  
**Reinforcement**

[LRFD Art. 5.8.3.4.2]

The  $\theta$  and  $\beta$  values are determined based on the strain in the flexural tension reinforcement. The strain in the reinforcement,  $\epsilon_x$ , is determined assuming that the section contains at least the minimum transverse reinforcement as specified in LRFD Art. 5.8.2.5.

$$\epsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p) \cot\theta - A_{ps}f_{po}}{2(E_s A_s + E_p A_{ps})} \leq 0.001$$

[LRFD Eq. 5.8.3.4.2-1]

where:

$$\begin{aligned} V_u &= \text{Applied factored shear force at the specified section, } 0.051L \\ &= 1.25(40.04 + 39.02 + 5.36) + 1.50(6.15) + 1.75(67.28 + 25.48) = 277.08 \text{ kips} \end{aligned}$$

$$\begin{aligned} M_u &= \text{Applied factored moment at the specified section, } 0.051L \\ &> V_u d_v \\ &= 1.25(233.54 + 227.56 + 31.29) + 1.50(35.84) + 1.75(291.58 + 116.33) \\ &= 1383.09 \text{ k-ft.} > 277.08(53.17/12) = 1227.69 \text{ k-ft. (O.K.)} \end{aligned}$$

$$\begin{aligned} N_u &= \text{Applied factored normal force at the specified section, } 0.051L = 0 \text{ kips} \end{aligned}$$

$$\begin{aligned} f_{po} &= \text{Parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi). For pretensioned members, LRFD Art. C5.8.3.4.2 indicates that } f_{po} \text{ can be taken as the stress in strands when the concrete is cast around them, which is jacking stress } f_{pj}, \text{ or } f_{pu}. \\ &= 0.75(270.0) = 202.5 \text{ ksi} \end{aligned}$$

$$\begin{aligned} V_p &= \text{Component of the effective prestressing force in the direction of the applied shear, kips} \\ &= (\text{force per strand})(\text{number of harped strands})(\sin\Psi) \end{aligned}$$

$$\Psi = \tan^{-1}\left(\frac{42.45}{49.4(12\text{in./ft.})}\right) = 0.072 \text{ rad. (see Figure A.2.7.3)}$$

$$V_p = 22.82(10) \sin(0.072) = 16.42 \text{ kips}$$

$$\epsilon_x = \frac{\frac{1383.09(12 \text{ in./ft.})}{53.17} + 0.5(277.08 - 16.42) \cot 23^\circ - 44(0.153)(202.5)}{2[28,000(0.0) + 28,500(44)(0.153)]}$$

$$\epsilon_x = -0.00194$$

Since this value is negative, LRFD Eq. 5.8.3.4.2-3 should be used to calculate  $\epsilon_x$ .

$$\epsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p) \cot \theta - A_{ps}f_{po}}{2(E_c A_c + E_s A_s + E_p A_{ps})}$$

where:

$A_c$  = Area of the concrete on the flexural tension side below  $h/2 = 473 \text{ in.}^2$

$E_c$  = Modulus of elasticity of girder concrete, ksi  
 $= 33,000(w_c)^{1.5} \sqrt{f'_c}$   
 $= [33,000(0.150)^{1.5} \sqrt{5.892}] = 4653.53 \text{ ksi}$

Strain in the flexural tension reinforcement is

$$\epsilon_x = \frac{\frac{1383.09(12 \text{ in./ft.})}{53.17} + 0.5(277.08 - 16.42) \cot 23^\circ - 44(0.153)(202.5)}{2[4653.53(473) + 28,000(0.0) + 28,500(44)(0.153)]}$$

$$\epsilon_x = -0.000155$$

Shear stress in the concrete is given as:

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} \quad [\text{LRFD Eq. 5.8.3.4.2-1}]$$

where:

$\phi$  = Resistance factor = 0.9 for shear in prestressed concrete members [LRFD Art. 5.5.4.2.1]

$$v_u = \frac{277.08 - 0.9(16.42)}{0.9(8.0)(53.17)} = 0.685 \text{ ksi}$$

$$v_u / f'_c = 0.685 / 5.892 = 0.12$$

### A.2.13.2.2 Values of $\beta$ and $\theta$

The values of  $\beta$  and  $\theta$  are determined using LRFD Table 5.8.3.4.2-1. Linear interpolation is allowed if the values lie between two rows.

Table A.2.13.1. Interpolation for  $\theta$  and  $\beta$  Values.

| $v_u / f'_c$ | $\epsilon_x \times 1000$ |          |               |
|--------------|--------------------------|----------|---------------|
|              | $\leq -0.200$            | $-0.155$ | $\leq -0.100$ |
| $\leq 0.100$ | 18.100                   |          | 20.400        |
|              | 3.790                    |          | 3.380         |
| 0.12         | 19.540                   | 20.47    | 21.600        |
|              | 3.302                    | 3.20     | 3.068         |
| $\leq 0.125$ | 19.900                   |          | 21.900        |
|              | 3.180                    |          | 2.990         |

$$\theta = 20.47^\circ > 23^\circ \text{ (assumed)}$$

Another iteration is made with  $\theta = 20.65^\circ$  to arrive at the correct value of  $\beta$  and  $\theta$ .

$d_e$  = Effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement = 56.45 in.

$d_v$  = Effective shear depth = 53.17 in.

The critical section near the support is greater of:

$d_v = 53.17$  in. and

$0.5d_v \cot \theta = 0.5(53.17)(\cot 20.47^\circ) = 71.2$  in. from the face of the support (controls)

Add half the bearing width (3.5 in.) to the critical section distance from the face of the support to get the distance of the critical section from the centerline of bearing.

Critical section for shear

$x = 71.2 + 3.5 = 74.7$  in. = 6.22 ft. (0.057L) from the centerline of bearing

Assuming the strain will be negative again, LRFD Eq. 5.8.3.4.2-3 will be used to calculate  $\epsilon_x$ .

$$\epsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p) \cot \theta - A_{ps}f_{po}}{2(E_c A_c + E_s A_s + E_p A_{ps})}$$

The shear forces and bending moments will be updated based on the updated critical section location.

$$\begin{aligned}
 V_u &= \text{Applied factored shear force at the specified section, } 0.057L \\
 &= 1.25(39.49 + 38.48 + 5.29) + 1.50(6.06) + 1.75(66.81 + 25.15) = 274.10 \text{ kips}
 \end{aligned}$$

$$\begin{aligned}
 M_u &= \text{Applied factored moment at the specified section, } 0.057L \\
 &> V_u d_v \\
 &= 1.25(260.18 + 253.53 + 34.86) + 1.50(39.93) + 1.75(324.63 + 129.60) \\
 &= 1540.50 \text{ k-ft.} > 274.10(53.17/12) = 1222.03 \text{ k-ft. (O.K.)}
 \end{aligned}$$

$$\epsilon_x = \frac{\frac{1540.50(12 \text{ in./ft.})}{53.17} + 0.5(274.10 - 16.42)\cot 20.47^\circ - 44(0.153)(202.5)}{2[4653.53(473) + 28,000(0.0) + 28,500(44)(0.153)]}$$

$$\epsilon_x = -0.000140$$

Shear stress in concrete

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} = \frac{274.10 - 0.9(16.42)}{0.9(8)(53.17)} = 0.677 \text{ ksi}$$

[LRFD Eq. 5.8.3.4.2-1]

$$v_u / f'_c = 0.677/5.892 = 0.115$$

Table A.2.13.2. Interpolation for  $\theta$  and  $\beta$  Values.

| $v_u / f'_c$ | $\epsilon_x \times 1000$ |          |               |
|--------------|--------------------------|----------|---------------|
|              | $\leq -0.200$            | $-0.140$ | $\leq -0.100$ |
| $\leq 0.100$ | 18.100                   |          | 20.40         |
|              | 3.790                    |          | 3.380         |
| 0.115        | 18.59                    | 20.22    | 21.30         |
|              | 3.424                    | 3.26     | 3.146         |
| $\leq 0.125$ | 19.90                    |          | 21.900        |
|              | 3.180                    |          | 2.990         |

$$\theta = 20.22^\circ \approx 20.47^\circ \text{ (from first iteration)}$$

Therefore, no further iteration is needed.

$$\beta = 3.26$$

**A.2.13.2.3**  
**Computation of**  
**Concrete**  
**Contribution**

The contribution of the concrete to the nominal shear resistance is given as:

$$V_c = 0.0316\beta\sqrt{f'_c} b_v d_v \quad [\text{LRFD Eq. 5.8.3.3-3}]$$

where:

$\beta$  = A factor indicating the ability of diagonally cracked concrete to transmit tension = 3.26

$f'_c$  = Compressive strength of concrete at service = 5.892 ksi

$b_v$  = Effective web width taken as the minimum web width within the depth  $d_v$ , in. = 8 in. (see Figure A.2.4.1)

$d_v$  = Effective shear depth = 53.17 in.

$$V_c = 0.0316(3.26)(\sqrt{5.892})(8.0)(53.17) = 106.36$$

**A.2.13.3**  
**Contribution of**  
**Reinforcement to**  
**Nominal Shear**  
**Resistance**  
**A.2.13.3.1**  
**Requirement for**  
**Reinforcement**

Check if  $V_u > 0.5\phi(V_c + V_p)$  [LRFD Eq. 5.8.2.4-1]

$$V_u = 274.10 \text{ kips} > 0.5(0.9)(106.36 + 16.42) = 55.25 \text{ kips}$$

Therefore, transverse shear reinforcement should be provided.

**A.2.13.3.2**  
**Required Area of**  
**Reinforcement**

The required area of transverse shear reinforcement is:

$$\frac{V_u}{\phi} \leq V_n = (V_c + V_s + V_p) \quad [\text{LRFD Eq. 5.8.3.3-1}]$$

where

$V_s$  = Shear force carried by transverse reinforcement

$$= \frac{V_u}{\phi} - V_c - V_p = \left( \frac{274.10}{0.9} - 106.36 - 16.42 \right) = 181.77 \text{ kips}$$

$$V_s = \frac{A_v f_y d_v (\cot \theta + \cot \alpha) \sin \alpha}{s} \quad [\text{LRFD Eq. 5.8.3.3-4}]$$

where

$A_v$  = Area of shear reinforcement within a distance  $s$ , in.<sup>2</sup>

$s$  = Spacing of stirrups, in.

$f_y$  = Yield strength of shear reinforcement, ksi

$\alpha$  = angle of inclination of transverse reinforcement to longitudinal axis =  $90^\circ$  for vertical stirrups

Therefore, area of shear reinforcement within a distance  $s$  is:

$$\begin{aligned} A_v &= (s V_s) / f_y d_v (\cot \theta + \cot \alpha) \sin \alpha \\ &= s(181.77) / (60)(53.17)(\cot 20.22^\circ + \cot 90^\circ) \sin 90^\circ = 0.021(s) \end{aligned}$$

If  $s = 12$  in., required  $A_v = 0.252$  in.<sup>2</sup>/ft.

### **A.2.13.3.3** **Determine spacing** **of reinforcement**

Check for maximum spacing of transverse reinforcement

[LRFD Art. 5.8.2.7]

check if  $v_u < 0.125 f'_c$

[LRFD Eq. 5.8.2.7-1]

or if  $v_u \geq 0.125 f'_c$

[LRFD Eq. 5.8.2.7-2]

$$0.125 f'_c = 0.125(5.892) = 0.74 \text{ ksi}$$

$$v_u = 0.677 \text{ ksi}$$

Since  $v_u < 0.125 f'_c$ , therefore,  $s \leq 24$  in. [LRFD Eq. 5.8.2.7-2]

$$s \leq 0.8 d_v = 0.8(53.17) = 42.54 \text{ in.}$$

Therefore maximum  $s = 24.0$  in.  $> s$  provided (O.K.)

Use #4 bar double legged stirrups at 12 in. c/c,

$$A_v = 2(0.20) = 0.40 \text{ in}^2/\text{ft} > 0.252 \text{ in}^2/\text{ft}$$

$$V_s = \frac{0.4(60)(53.17)(\cot 20.47^\circ)}{12} = 283.9 \text{ kips}$$



**A.2.11.3.4**  
**Minimum**  
**Reinforcement**  
**requirement**

The area of transverse reinforcement should not be less than:  
 [LRFD Art. 5.8.2.5]

$$0.0316 \sqrt{f'_c} \frac{b_v s}{f_y} \quad \text{[LRFD Eq. 5.8.2.5-1]}$$

$$= 0.0316 \sqrt{5.892} \frac{(8)(12)}{60} = 0.12 < A_v \text{ provided} \quad (\text{O.K.})$$

**A.2.13.5**  
**Maximum Nominal**  
**Shear Resistance**

In order to assure that the concrete in the web of the girder will not crush prior to yield of the transverse reinforcement, the LRFD Specifications give an upper limit for  $V_n$  as follows:

$$V_n = 0.25 f'_c b_v d_v + V_p \quad \text{[LRFD Eq. 5.8.3.3-2]}$$

Comparing above equation with LRFD Eq. 5.8.3.3-1

$$V_c + V_s \leq 0.25 f'_c b_v d_v$$

$$106.36 + 283.9 = 390.26 \text{ kips} \leq 0.25(5.892)(8)(53.17)$$

$$= 626.55 \text{ kips} \quad \text{O.K.}$$

This is a sample calculation for determining transverse reinforcement requirement at critical section and this procedure can be followed to find the transverse reinforcement requirement at increments along the length of the girder.

**A.2.14**  
**INTERFACE SHEAR**  
**TRANSFER**  
**A.2.12.1**  
**Factored Horizontal**  
**Shear**

[LRFD Art. 5.8.4]

At the strength limit state, the horizontal shear at a section can be calculated as follows

$$V_h = \frac{V_u}{d_v} \quad [\text{LRFD Eq. C5.8.4.1-1}]$$

where

$V_h$  = Horizontal shear per unit length of the girder, kips

$V_u$  = Factored shear force at specified section due to superimposed loads, kips

$d_v$  = Distance between resultants of tensile and compressive forces ( $d_e - a/2$ ), in.

The LRFD Specifications do not identify the location of the critical section. For convenience, it will be assumed here to be the same location as the critical section for vertical shear, at point 0.057L

Using load combination Strength I:

$$V_u = 1.25(5.29) + 1.50(6.06) + 1.75(66.81 + 25.15) = 176.63 \text{ kips}$$

$$d_v = 53.17 \text{ in}$$

Therefore applied factored horizontal shear is:

$$V_h = \frac{176.63}{53.17} = 3.30 \text{ kips/in.}$$

$$\text{Required } V_n = V_h / \phi = 3.30 / 0.9 = 3.67 \text{ kip/in.}$$

**A.2.14.2**  
**Required Nominal**  
**Resistance**

The nominal shear resistance of the interface surface is:

$$V_n = cA_{cv} + \mu [A_{vf}f_y + P_c] \quad [\text{LRFD Eq. 5.8.4.1-1}]$$

where

$c$  = Cohesion factor [LRFD Art. 5.8.4.2]

$\mu$  = Friction factor [LRFD Art. 5.8.4.2]

$A_{cv}$  = Area of concrete engaged in shear transfer, in<sup>2</sup>.

$A_{vf}$  = Area of shear reinforcement crossing the shear plane, in<sup>2</sup>.

$P_c$  = Permanent net compressive force normal to the shear plane, kips

$f_y$  = Shear reinforcement yield strength, ksi

### **A.2.14.3**

#### **Required Interface Shear Reinforcement**

For concrete placed against clean, hardened concrete and free of laitance, but not an intentionally roughened surface:

[LRFD Art. 5.8.4.2]

$c = 0.075$  ksi

$\mu = 0.6\lambda$ , where  $\lambda = 1.0$  for normal weight concrete, and therefore,

$\mu = 0.6$

The actual contact width,  $b_v$ , between the slab and the girder is 20 in.

$A_{cv} = (20 \text{ in.})(1 \text{ in.}) = 20 \text{ in.}^2$

The LRFD Eq. 5.8.4.1-1 can be solved for  $A_{vf}$  as follows:

$3.67 = (0.075)(20) + 0.6(A_{vf}(60) + 0)$

Solving for  $A_{vf} = 0.06 \text{ in.}^2/\text{in.}$  or  $0.72 \text{ in.}^2/\text{ft.}$

2 - #4 double-leg bar per ft are provided.

Area of steel provided = 2 (0.40) = 0.80 in.<sup>2</sup> / ft.

Provide 2 legged #4 bars at 6 in. c/c

The web reinforcement shall be provided at 6 in. c/c which can be extended into the cast-in-place slab to account for the interface shear requirement.

### **A.2.14.3.1**

#### **Minimum Interface shear reinforcement**

Minimum  $A_{vf} \geq (0.05b_v)/f_y$  [LRFD Eq. 5.8.4.1-4]

where  $b_v$  = width of the interface

$A_{vf} = 0.80 \text{ in.}^2/\text{ft.} > [0.05(20)/60](12 \text{ in.}/\text{ft.}) = 0.2 \text{ in.}^2/\text{ft.}$  O.K.

$V_n \text{ provided} = 0.075(20) + 0.6 \left( \frac{0.80}{12} (60) + 0 \right) = 3.9 \text{ kips/in.}$

$0.2 f'_c A_{cv} = 0.2(4.0)(20) = 16 \text{ kips/in.}$

$0.8A_{cv} = 0.8(20) = 16 \text{ kips/in.}$

Since provided  $V_n \leq 0.2 f'_c A_{cv}$  O.K. [LRFD Eq. 5.8.4.1-2]

$\leq 0.8 A_{cv}$  O.K. [LRFD Eq. 5.8.4.1-3]

**A.2.15**  
**MINIMUM**  
**LONGITUDINAL**  
**REINFORCEMENT**  
**REQUIREMENT**

[LRFD Art. 5.8.3.5]

Longitudinal reinforcement should be proportioned so that at each section the following equation is satisfied

$$A_s f_y + A_{ps} f_{ps} \geq \frac{M_u}{d_v \phi} + 0.5 \frac{N_u}{\phi} + \left( \frac{V_u}{\phi} - 0.5 V_s - V_p \right) \cot \theta$$

[LRFD Eq. 5.8.3.5-1]

where

$A_s$  = Area of non prestressed tension reinforcement, in.<sup>2</sup>

$f_y$  = Specified minimum yield strength of reinforcing bars, ksi

$A_{ps}$  = Area of prestressing steel at the tension side of the section, in.<sup>2</sup>

$f_{ps}$  = Average stress in prestressing steel at the time for which the nominal resistance is required, ksi

$M_u$  = Factored moment at the section corresponding to the factored shear force, kip-ft.

$N_u$  = Applied factored axial force, kips

$V_u$  = Factored shear force at the section, kips

$V_s$  = Shear resistance provided by shear reinforcement, kips

$V_p$  = Component in the direction of the applied shear of the effective prestressing force, kips

$d_v$  = Effective shear depth, in.

$\theta$  = Angle of inclination of diagonal compressive stresses.

**A.2.15.1**  
**Required**  
**Reinforcement at**  
**Face of Bearing**

[LRFD Art. 5.8.3.5]

Width of bearing = 7.0 in.

Distance of section =  $7/2 = 3.5$  in. = 0.291 ft.

Shear forces and bending moment are calculated at this section

$$V_u = 1.25(44.35 + 43.22 + 5.94) + 1.50(6.81) + 1.75(71.05 + 28.14) \\ = 300.69 \text{ kips.}$$

$$M_u = 1.25(12.04 + 11.73 + 1.61) + 1.50(1.85) + 1.75(15.11 + 6.00) \\ = 71.44 \text{ Kip-ft.}$$

$$\frac{M_u}{d_v \phi} + 0.5 \frac{N_u}{\phi} + \left( \frac{V_u}{\phi} - 0.5V_s - V_p \right) \cot \theta \\ = \frac{71.44(12 \text{ in./ft.})}{53.17(0.9)} + 0 + \left( \frac{300.69}{0.90} - 0.5(283.9) - 16.42 \right) \cot 20.47^\circ \\ = 484.09 \text{ kips}$$

The crack plane crosses the centroid of the 44 straight strands at a distance of  $6 + 5.33 \cot 20.47^\circ = 20.14$  in. from the end of the girder.

Since the transfer length is 30 in. the available prestress from 44 straight strands is a fraction of the effective prestress,  $f_{pe}$ , in these strands. The 10 harped strands do not contribute the tensile capacity since they are not on the flexural tension side of the member.

Therefore available prestress force is:

$$A_s f_y + A_{ps} f_{ps} = 0 + 44(0.153) \left( 149.18 \frac{20.33}{30} \right) = 680.57 \text{ kips}$$

$$A_s f_y + A_{ps} f_{ps} = 649.63 \text{ kips} > 484.09 \text{ kips}$$

Therefore additional longitudinal reinforcement is not required.

[LRFD Art. 5.10.10]

**A.2.16**  
**PRETENSIONED**  
**ANCHORAGE ZONE**  
**A.2.16.1**  
**Minimum Vertical**  
**Reinforcement**

[LRFD Art. 5.10.10.1]

Design of the anchorage zone reinforcement is computed using the force in the strands just prior to transfer:

Force in the strands at transfer  
 $F_{pi} = 54(0.153)(202.5) = 1673.06 \text{ kips}$

The bursting resistance,  $P_r$ , should not be less than 4 percent of  $F_{pi}$   
 [LRFD Arts. 5.10.10.1 and C3.4.3]

$$P_r = f_s A_s \geq 0.04 F_{pi} = 0.04(1673.06) = 66.90 \text{ kips}$$

where

$A_s$  = Total area of vertical reinforcement located within a distance of  $h/4$  from the end of the girder, in<sup>2</sup>.

$f_s$  = Stress in steel not exceeding 20 ksi.

Solving for required area of steel  $A_s = 66.90/20 = 3.35 \text{ in}^2$

Atleast  $3.35 \text{ in}^2$  of vertical transverse reinforcement should be provided within a distance of ( $h/4 = 62 / 4 = 15.5 \text{ in}$ ). from the end of the girder.

Use 6 - #5 double leg bars at 2.0 in. spacing starting at 2 in. from the end of the girder.

The provided  $A_s = 6(2)0.31 = 3.72 \text{ in}^2 > 3.35 \text{ in}^2$  O.K.

**A.2.16.2**  
**Confinement**  
**Reinforcement**

[LRFD Art. 5.10.10.2]

For a distance of  $1.5d = 1.5(54) = 81 \text{ in}$ . from the end of the girder, reinforcement is placed to confine the prestressing steel in the bottom flange. The reinforcement shall not be less than #3 deformed bars with spacing not exceeding 6 in. The reinforcement should be of shape which will confine the strands.

**A.2.17**  
**CAMBER AND**  
**DEFLECTIONS**  
**A.2.17.1**  
**Maximum Camber**

The LRFD Specifications do not provide any guidelines for the determination of camber of prestressed concrete members. The Hyperbolic Functions Method proposed by Rauf and Furr (1970) for the calculation of maximum camber is used by TxDOT's prestressed concrete bridge design software, PSTRS14 (TxDOT 2004). The following steps illustrate the Hyperbolic Functions method for the estimation of maximum camber.

Step 1: The total prestressing force after initial prestress loss due to elastic shortening has occurred

$$P = \frac{P_i}{\left(1 + pn + \frac{e_c^2 A_s n}{I}\right)} + \frac{M_D e_c A_s n}{I \left(1 + pn + \frac{e_c^2 A_s n}{I}\right)}$$

where:

$$\begin{aligned} P_i &= \text{Anchor force in prestressing steel} \\ &= (\text{number of strands})(\text{area of strand})(f_{si}) \\ P_i &= 54(0.153)(202.5) = 1673.06 \text{ kips} \end{aligned}$$

$$f_{pi} = \text{Before transfer, } \leq 0.75 f_{pu} = 202,500 \text{ psi}$$

[LRFD Table 5.9.3-1]

$$f_{pu} = \text{Ultimate strength of prestressing strands} = 270 \text{ ksi}$$

$$f_{pi} = 0.75(270) = 202.5 \text{ ksi}$$

$$\begin{aligned} I &= \text{Moment of inertia of the non-composite precast girder} \\ &= 260403 \text{ in.}^4 \end{aligned}$$

$$e_c = \text{Eccentricity of prestressing strands at the midspan} \\ = 19.12 \text{ in.}$$

$$M_D = \text{Moment due to self-weight of the girder at midspan} \\ = 1209.98 \text{ k-ft.}$$

$$A_s = \text{Area of prestressing steel} \\ = (\text{number of strands})(\text{area of strand}) \\ = 54(0.153) = 8.262 \text{ in.}^2$$

$$p = A_s/A$$

$$A = \text{Area of girder cross-section} = 788.4 \text{ in.}^2$$

$$p = \frac{8.262}{788.4} = 0.0105$$

$$n = \text{Modular ratio between prestressing steel and the girder concrete at release} = E_s/E_{ci}$$

$$E_{ci} = \text{Modulus of elasticity of the girder concrete at release} \\ = 33(w_c)^{3/2} \sqrt{f'_{ci}} \quad [\text{STD Eq. 9-8}]$$

$$w_c = \text{Unit weight of concrete} = 150 \text{ pcf}$$

$$f'_{ci} = \text{Compressive strength of precast girder concrete at release} = 5,892 \text{ psi}$$

$$E_{ci} = [33(150)^{3/2} \sqrt{5,892}] \left( \frac{1}{1,000} \right) = 4,653.53 \text{ ksi}$$

$$E_s = \text{Modulus of elasticity of prestressing strands} \\ = 28,000 \text{ ksi}$$

$$n = 28,000/4,653.53 = 6.12$$

$$\left( 1 + pn + \frac{e_c^2 A_s n}{I} \right) = 1 + (0.0105)(6.12) + \frac{(19.12^2)(8.262)(6.12)}{260,403} \\ = 1.135$$

$$P = \frac{1,673.06}{1.135} + \frac{(1,209.98)(12 \text{ in./ft.})(19.12)(8.262)(6.12)}{260,403(1.135)}$$

$$= 1474.06 + 47.49 = 1521.55 \text{ kips}$$



Initial prestress loss is defined as

$$PL_i = \frac{P_i - P}{P_i} = \frac{1,673.06 - 1521.55}{1,673.06} = 0.091 = 9.1\%$$

The stress in the concrete at the level of the centroid of the prestressing steel immediately after transfer is determined as follows.

$$f_{ci}^s = P \left( \frac{1}{A} + \frac{e_c^2}{I} \right) - f_c^s$$

where:

$$\begin{aligned} f_c^s &= \text{Concrete stress at the level of centroid of prestressing steel due to dead loads, ksi} \\ &= \frac{M_D e_c}{I} = \frac{(1,209.98)(12 \text{ in./ft.})(19.12)}{260,403} = 1.066 \text{ ksi} \end{aligned}$$

$$f_{ci}^s = 1521.55 \left( \frac{1}{788.4} + \frac{19.12^2}{260,403} \right) - 1.066 = 3.0 \text{ ksi}$$

The ultimate time dependent prestress loss is dependent on the ultimate creep and shrinkage strains. As the creep strains vary with the concrete stress, the following steps are used to evaluate the concrete stresses and adjust the strains to arrive at the ultimate prestress loss. It is assumed that the creep strain is proportional to the concrete stress and the shrinkage stress is independent of concrete stress. (Sinno 1970)

Step 2: Initial estimate of total strain at steel level assuming constant sustained stress immediately after transfer

$$\epsilon_{ci}^s = \epsilon_{cr}^\infty f_{ci}^s + \epsilon_{sh}^\infty$$

where:

$$\epsilon_{cr}^\infty = \text{Ultimate unit creep strain} = 0.00034 \text{ in./in. [this value is prescribed by Sinno et. al. (1970)]}$$

$\epsilon_{sh}^{\infty}$  = Ultimate unit shrinkage strain = 0.000175 in./in. [this value is prescribed by Sinno et. al. (1970)]

$$\epsilon_{c1}^s = 0.00034(3.0) + 0.000175 = 0.001195 \text{ in./in.}$$

Step 3: The total strain obtained in Step 2 is adjusted by subtracting the elastic strain rebound as follows

$$\epsilon_{c2}^s = \epsilon_{c1}^s - \epsilon_{c1}^s E_s \frac{A_s}{E_{ci}} \left( \frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\begin{aligned} \epsilon_{c2}^s &= 0.001195 - 0.001195 (28,500) \frac{8.262}{4,653.53} \left( \frac{1}{788.4} + \frac{19.12^2}{260,403} \right) \\ &= 0.001033 \text{ in./in.} \end{aligned}$$

Step 4: The change in concrete stress at the level of centroid of prestressing steel is computed as follows:

$$\Delta f_c^s = \epsilon_{c2}^s E_s A_s \left( \frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\Delta f_c^s = 0.001033 (28,500)(8.262) \left( \frac{1}{788.4} + \frac{19.12^2}{260,403} \right) = 0.648 \text{ ksi}$$

Step 5: The total strain computed in Step 2 needs to be corrected for the change in the concrete stress due to creep and shrinkage strains.

$$\epsilon_{c4}^s = \epsilon_{cr}^{\infty} \left( f_{ci}^s - \frac{\Delta f_c^s}{2} \right) + \epsilon_{sh}^{\infty}$$

$$\epsilon_{c4}^s = 0.00034 \left( 3.0 - \frac{0.648}{2} \right) + 0.000175 = 0.001085 \text{ in./in.}$$

Step 6: The total strain obtained in Step 5 is adjusted by subtracting the elastic strain rebound as follows

$$\epsilon_{c5}^s = \epsilon_{c4}^s - \epsilon_{c4}^s E_s \frac{A_s}{E_{ci}} \left( \frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\begin{aligned} \epsilon_{c5}^s &= 0.001085 - 0.001085(28500) \frac{8.262}{4653.53} \left( \frac{1}{788.4} + \frac{19.12^2}{260403} \right) \\ &= 0.000938 \text{ in./in} \end{aligned}$$

Sinno (1970) recommends stopping the updating of stresses and adjustment process after Step 6. However, as the difference between the strains obtained in Steps 3 and 6 is not negligible, this process is carried on until the total strain value converges.

Step 7: The change in concrete stress at the level of centroid of prestressing steel is computed as follows:

$$\Delta f_{c1}^s = \epsilon_{c5}^s E_s A_s \left( \frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\Delta f_{c1}^s = 0.000938(28,500)(8.262) \left( \frac{1}{788.4} + \frac{19.12^2}{260,403} \right) = 0.5902 \text{ ksi}$$

Step 8: The total strain computed in Step 5 needs to be corrected for the change in the concrete stress due to creep and shrinkage strains.

$$\epsilon_{c6}^s = \epsilon_{cr}^\infty \left( f_{ci}^s - \frac{\Delta f_{c1}^s}{2} \right) + \epsilon_{sh}^\infty$$

$$\epsilon_{c6}^s = 0.00034 \left( 3.0 - \frac{0.5902}{2} \right) + 0.000175 = 0.001095 \text{ in./in.}$$

Step 9: The total strain obtained in Step 8 is adjusted by subtracting the elastic strain rebound as follows

$$\epsilon_{c7}^s = \epsilon_{c6}^s - \epsilon_{c6}^s E_s \frac{A_s}{E_{ci}} \left( \frac{1}{A} + \frac{e_c^2}{I} \right)$$

$$\begin{aligned} \epsilon_{c7}^s &= 0.001095 - 0.001095(28,500) \frac{8.262}{4,653.53} \left( \frac{1}{788.4} + \frac{19.12^2}{260,403} \right) \\ &= 0.000947 \text{ in./in} \end{aligned}$$

The strains have sufficiently converged and no more adjustments are needed.

Step 10: Computation of final prestress loss

Time dependent loss in prestress due to creep and shrinkage strains is given as

$$PL^\infty = \frac{\epsilon_{c7}^s E_s A_s}{P_i} = \frac{0.000947(28,500)(8.262)}{1,673.06} = 0.133 = 13.3\%$$

Total final prestress loss is the sum of initial prestress loss and the time dependent prestress loss expressed as follows

$$PL = PL_i + PL^\infty$$

where:

$PL$  = Total final prestress loss percent.

$PL_i$  = Initial prestress loss percent = 9.1%

$PL^\infty$  = Time dependent prestress loss percent = 13.3%

$$PL = 9.1 + 13.3 = 22.4\%$$

Step 11: The initial deflection of the girder under self-weight is calculated using the elastic analysis as follows:

$$C_{DL} = \frac{5 w L^4}{384 E_{ci} I}$$

where:

$C_{DL}$  = Initial deflection of the girder under self-weight, ft.

$w$  = Self-weight of the girder = 0.821 kips/ft.

$L$  = Total girder length = 109.67 ft.

$E_{ci}$  = Modulus of elasticity of the girder concrete at release  
= 4,653.53 ksi = 670,108.32 k/ft.<sup>2</sup>

$I$  = Moment of inertia of the non-composite precast girder  
= 260403 in.<sup>4</sup> = 12.558 ft.<sup>4</sup>

$$C_{DL} = \frac{5(0.821)(109.67^4)}{384(670,108.32)(12.558)} = 0.184 \text{ ft.} = 2.208 \text{ in.}$$

Step 12: Initial camber due to prestress is calculated using the moment area method. The following expression is obtained from the M/EI diagram to compute the camber resulting from the initial prestress.

$$C_{pi} = \frac{M_{pi}}{E_{ci} I}$$

where:

$$M_{pi} = [0.5(P) (e_e) (0.5L)^2 + 0.5(P) (e_c - e_e) (0.67) (HD)^2 + 0.5P (e_c - e_e) (HD_{dis}) (0.5L + HD)] / (Eci)(I)$$

$P$  = Total prestressing force after initial prestress loss due to elastic shortening have occurred = **1521.55 kips**

$HD$  = Hold-down distance from girder end  
= 49.404 ft. = 592.85 in. (see Figure A.1.7.3)

$HD_{dis}$  = Hold-down distance from the center of the girder span  
=  $0.5(109.67) - 49.404 = 5.431$  ft. = 65.17 in.

$e_e$  = Eccentricity of prestressing strands at girder end  
= 11.34 in.

$e_c$  = Eccentricity of prestressing strands at midspan  
= 19.12 in.

$L$  = Overall girder length = 109.67 ft. = 1,316.04 in.

$$M_{pi} = \{0.5(1521.55) (11.34) [(0.5) (1,316.04)]^2 + 0.5(1521.55) (19.12 - 11.34) (0.67) (592.85)^2 + 0.5(1521.55) (19.12 - 11.34) (65.17)[0.5(1,316.04) + 592.85]\}$$

$$M_{pi} = 3.736 \times 10^9 + 1.394 \times 10^9 + 0.483 \times 10^9 = 5.613 \times 10^9$$

$$C_{pi} = \frac{5.613 \times 10^9}{(4,653.53)(260,403)} = 4.63 \text{ in.} = 0.386 \text{ ft.}$$

Step 13: The initial camber,  $C_i$ , is the difference between the upward camber due to initial prestressing and the downward deflection due to self-weight of the girder.

$$C_i = C_{pi} - C_{DL} = 4.63 - 2.208 = 2.422 \text{ in.} = 0.202 \text{ ft.}$$

Step 14: The ultimate time-dependent camber is evaluated using the following expression.

$$\text{Ultimate camber } C_t = C_i (1 - PL^\infty) \frac{\varepsilon_{cr}^\infty \left( f_{ci}^s - \frac{\Delta f_{cl}^s}{2} \right) + \varepsilon_e^s}{\varepsilon_e^s}$$

where:

$$\varepsilon_e^s = \frac{f_{ci}^s}{E_{ci}} = \frac{3.0}{4,653.53} = 0.000619 \text{ in./in.}$$

$$C_t = 2.422(1 - 0.133) \frac{0.00034 \left( 3.0 - \frac{0.5902}{2} \right) + 0.000645}{0.000645}$$

$$C_t = 5.094 \text{ in.} = 0.425 \text{ ft. } \uparrow$$

### A.2.17.2 Deflection due to Slab Weight

The deflection due to the slab weight is calculated using an elastic analysis as follows.

Deflection of the girder at midspan

$$\Delta_{slabI} = \frac{5 w_s L^4}{384 E_c I}$$

where:

$$w_s = \text{Weight of the slab} = 0.80 \text{ kips/ft.}$$

$$\begin{aligned} E_c &= \text{Modulus of elasticity of girder concrete at service} \\ &= 33(w_c)^{3/2} \sqrt{f'_c} \\ &= 33(150)^{1.5} \sqrt{5,892} \left( \frac{1}{1,000} \right) = 4,653.53 \text{ ksi} \end{aligned}$$

$$\begin{aligned} I &= \text{Moment of inertia of the non-composite girder section} \\ &= 260,403 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} L &= \text{Design span length of girder (center to center bearing)} \\ &= 108.583 \text{ ft.} \end{aligned}$$

$$\begin{aligned} \Delta_{slabI} &= \frac{5 \left( \frac{0.80}{12 \text{ in./ft.}} \right) [(108.583)(12 \text{ in./ft.})]^4}{384(4,653.53)(260,403)} \\ &= 2.06 \text{ in.} = 0.172 \text{ ft. } \downarrow \end{aligned}$$

Deflection at quarter span due to slab weight

$$\Delta_{slab2} = \frac{57 w_s L^4}{6144 E_c I}$$

$$\Delta_{slab2} = \frac{57 \left( \frac{0.80}{12 \text{ in./ft.}} \right) [(108.583)(12 \text{ in./ft.})]^4}{6,144(4,653.53)(260,403)}$$

$$= 1.471 \text{ in.} = 0.123 \text{ ft.} \downarrow$$

### **A.2.17.3** **Deflections due to** **Superimposed Dead** **Loads**

Deflection due to barrier weight at midspan

$$\Delta_{barr1} = \frac{5 w_{barr} L^4}{384 E_c I_c}$$

where:

$$w_{barr} = \text{Weight of the barrier} = 0.109 \text{ kips/ft.}$$

$$I_c = \text{Moment of inertia of composite section} = 651,886.0 \text{ in}^4$$

$$\Delta_{barr1} = \frac{5 \left( \frac{0.109}{12 \text{ in./ft.}} \right) [(108.583)(12 \text{ in./ft.})]^4}{384(4,653.53)(651,886.0)}$$

$$= 0.141 \text{ in.} = 0.0118 \text{ ft.} \downarrow$$

Deflection at quarter span due to barrier weight

$$\Delta_{barr2} = \frac{57 w_{barr} L^4}{6144 E_c I_c}$$

$$\Delta_{barr2} = \frac{57 \left( \frac{0.109}{12 \text{ in./ft.}} \right) [(108.583)(12 \text{ in./ft.})]^4}{6,144(4,653.53)(651,886.0)}$$

$$= 0.08 \text{ in.} = 0.0067 \text{ ft.} \downarrow$$

Deflection due to wearing surface weight at midspan

$$\Delta_{ws1} = \frac{5 w_{ws} L^4}{384 E_c I_c}$$

where

$$w_{ws} = \text{Weight of wearing surface} = 0.128 \text{ kips/ft.}$$

$$\Delta_{ws1} = \frac{5 \left( \frac{0.128}{12 \text{ in./ft.}} \right) [(108.583)(12 \text{ in./ft.})]^4}{384(4,653.53)(651,886.0)}$$

$$= 0.132 \text{ in.} = 0.011 \text{ ft.} \downarrow$$

Deflection at quarter span due to wearing surface

$$\Delta_{ws2} = \frac{57 w_{ws} L^4}{6144 E_c I}$$

$$\Delta_{ws2} = \frac{57 \left( \frac{0.128}{12 \text{ in./ft.}} \right) [(108.583)(12 \text{ in./ft.})]^4}{6,144(4,529.66)(657,658.4)}$$

$$= 0.094 \text{ in.} = 0.0078 \text{ ft.} \downarrow$$

#### **A.2.17.4**

#### **Total Deflection due to Dead Loads**

The total deflection at midspan due to slab weight and superimposed loads is:

$$\Delta_{T1} = \Delta_{slab1} + \Delta_{barr1} + \Delta_{ws1}$$

$$= 0.172 + 0.0118 + 0.011 = 0.1948 \text{ ft.} \downarrow$$

The total deflection at quarter span due to slab weight and superimposed loads is:

$$\Delta_{T2} = \Delta_{slab2} + \Delta_{barr2} + \Delta_{ws2}$$

$$= 0.123 + 0.0067 + 0.0078 = 0.1375 \text{ ft.} \downarrow$$

The deflections due to live loads are not calculated in this example as they are not a design factor for TxDOT bridges.



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